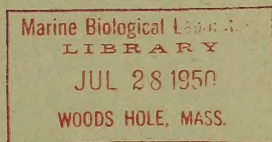
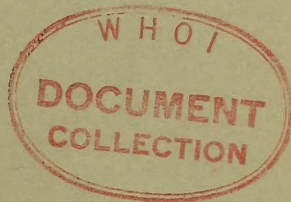


Vol. 4, No. 3

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS



THE
BULLETIN
OF THE
BEACH EROSION BOARD
OFFICE, CHIEF OF ENGINEERS
WASHINGTON, D.C.



TC
203
B84
v.4n.3

VOL. 4

JULY 1, 1950

NO. 3

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

THE BULLETIN OF THE BEACH EROSION BOARD

TABLE OF CONTENTS

	Page
Southern Hemisphere Swell and Waves From A Tropical Storm at Long Beach, California	1
The Wind Element in Beach Erosion	19
The Lag and Reduction of Range in Tide Gage Wells	24
Beach Erosion Studies	41
Beach Erosion Literature	50



DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

THE BULLETIN OF THE BEACH EROSION BOARD

TABLE OF CONTENTS

	Page
Southern Hemisphere Swell and Waves From A Tropical Storm at Long Beach, California	1
The Wind Element in Beach Erosion	19
The Lag and Reduction of Range in Tide Gage Wells	24
Beach Erosion Studies	41
Beach Erosion Literature	50



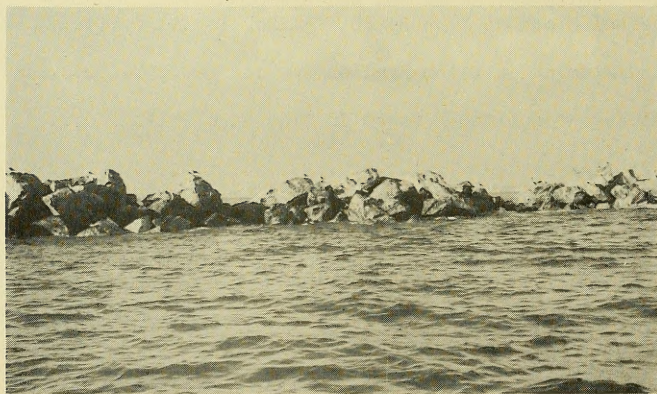


**FIGURE 1. THE RELATIVELY UNDAMAGED WESTERN PART AND THE
DAMAGED EASTERN PART OF THE LOS ANGELES-LONG BEACH DETACHED
BREAKWATER AFTER WAVE ACTION OF SEPTEMBER 1939.**

SOUTHERN HEMISPHERE SWELL AND WAVES FROM A TROPICAL STORM AT LONG BEACH, CALIFORNIA

This paper is published as a companion piece to the paper, "Wave Refraction at Long Beach and Santa Barbara, California", by M. P. O'Brien, published in the Beach Erosion Board Bulletin, Vol. 4, No. 1, January 1950. The paper was prepared by Paul L. Horrer and represents results of research carried out for the Hydrographic Office and the Office of Naval Research of the Department of the Navy under contract with the University of California. Scripps Contribution No. 475.

Introduction

On two occasions nine years apart large waves entered San Pedro Bay and caused extensive damage to the harbor breakwaters in the Long Beach-San Pedro area. During the period April 20 to 24, 1930, intensive wave action resulted in the displacement of 20,000 tons of superstructure stone from the Long Beach breakwater. On September 24 and 25, 1939, large waves caused some damage to the San Pedro Breakwater and great damage to the Los Angeles-Long Beach Detached Breakwater (Figure 1). The present study was undertaken in an attempt to analyze the meteorological conditions which produced the damaging wave action in this region and to provide information which may be used in planning future modification and improvement of the breakwaters.

As indicated by O'Brien (1947) it seems impossible that waves from westerly directions could bring sufficient concentration of wave energy to effect material damage to these breakwaters. Except for a narrow exposed western sector between Santa Rosa and Santa Barbara Islands all swell from the west must undergo diffraction and such great refraction around islands that little energy can reach this portion of San Pedro Bay. The only unsheltered directions of approach for swell lie within a wider sector to the south between the coast and the islands of Santa Catalina and San Clemente. Two distinct types of waves which could damage the breakwaters may enter through this exposed southern sector. These are swell propagated from storms in the southern hemisphere and wind waves or swell produced by northern hemisphere storms located south of Long Beach. It is necessary to stress the importance of each type separately.

The synoptic weather maps for the critical period in April, 1930 do not reveal any northern hemisphere storm which could generate large waves or swell which would pass through the exposed southern sector. This leaves only the possibility of a long, low swell from a storm in the southern hemisphere. Refraction diagrams (Figures 2-5) have been constructed for San Pedro Bay according to the procedure given by Munk and Traylor (1947). These diagrams indicate that San Pedro Shelf, the submarine plateau due

south of Long Beach, very effectively converged swell from the south upon certain portions of the Long Beach Breakwater, which with the San Pedro Breakwater formed the two principal structures protecting the harbor in 1930.

The intensive wave action which resulted in damage to the breakwaters in 1939 was derived from a tropical storm in the northern hemisphere. The characteristics of these waves differed materially from those of the swell which caused the damage in 1930. Therefore, the remainder of this discussion is divided into two parts: (1) southern hemisphere swell, and (2) waves from a tropical storm.

Part I - Southern Hemisphere Swell

The refraction of southern hemisphere swell has been examined for directions of approach of 170° , 180° , and 190° using a wave period of 20 seconds. For all three directions refraction diagrams were first constructed on small-scale charts showing San Clemente and Santa Catalina Islands and the approaches to San Pedro Bay (Figure 2). This type of presentation gives an overall picture of the bay's protection from, or exposure to, swell from given directions. Figure 2 is drawn for 190° , which is approximately the westernmost limit of the exposed southern sector from which southern hemisphere swell can converge upon the breakwaters. From directions west of 190° the zone of convergence shifts to the east of the outer breakwater.

The influence of the islands upon swell from direction 190° is illustrated by the extreme divergence of orthogonals a and b in Figure 2. In contrast to this, the portions of the wave crest unaffected by the islands between orthogonals c and e converge in crossing San Pedro Shelf. The waves passing both sides of the islands are refracted around the islands and meet to form a confused, crisscross pattern in the lee. Only the crests of the waves traveling around the east side of the islands have been drawn up to the breakwater. The waves traveling around the west side experience much greater divergence and contain negligible intensity of energy after refraction.

The boundary around San Pedro Bay in Figure 2 denotes the area covered by the detailed refraction diagrams presented in Figures 3-5. These large-scale diagrams were constructed using a crest interval of one wave period. The wave crests have been completed only up to the outer breakwater. In order to show quantitatively the intensity of wave energy for the wave crests at the breakwater and adjacent beaches bar graphs have been drawn. The lengths of the bars are proportional to the refraction factors according to the scale in the legend. The refraction factors have been computed assuming breaking waves only where the bars are located on the beaches west and east of the breakwater.

The detailed refraction diagram for 190° (Figure 3) shows divergence at the western part of the outer breakwater resulting primarily from the effect of refraction around San Clemente and Santa Catalina Islands. This island effect is accentuated by refraction over San Pedro Sea Valley, the

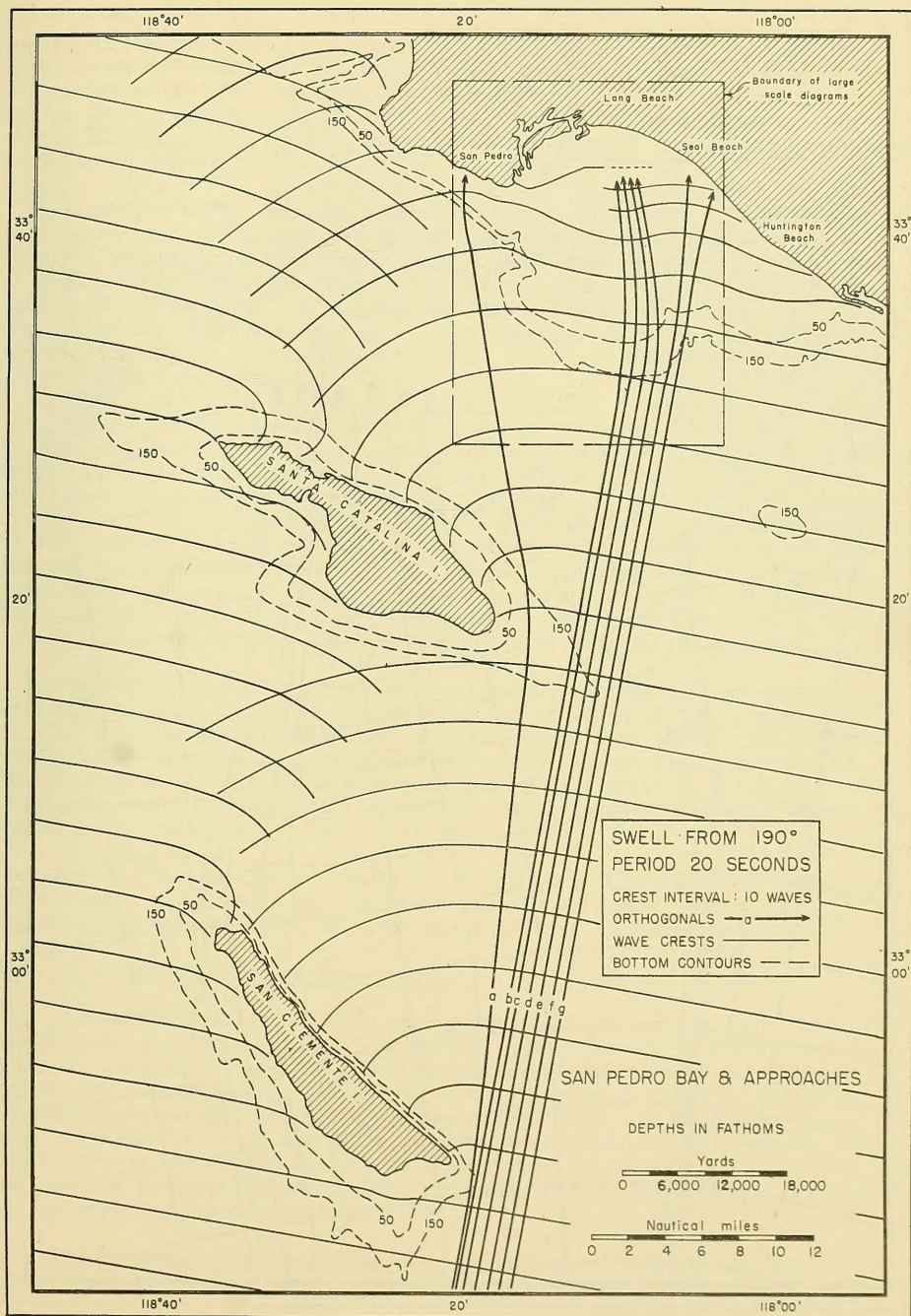


FIGURE 2

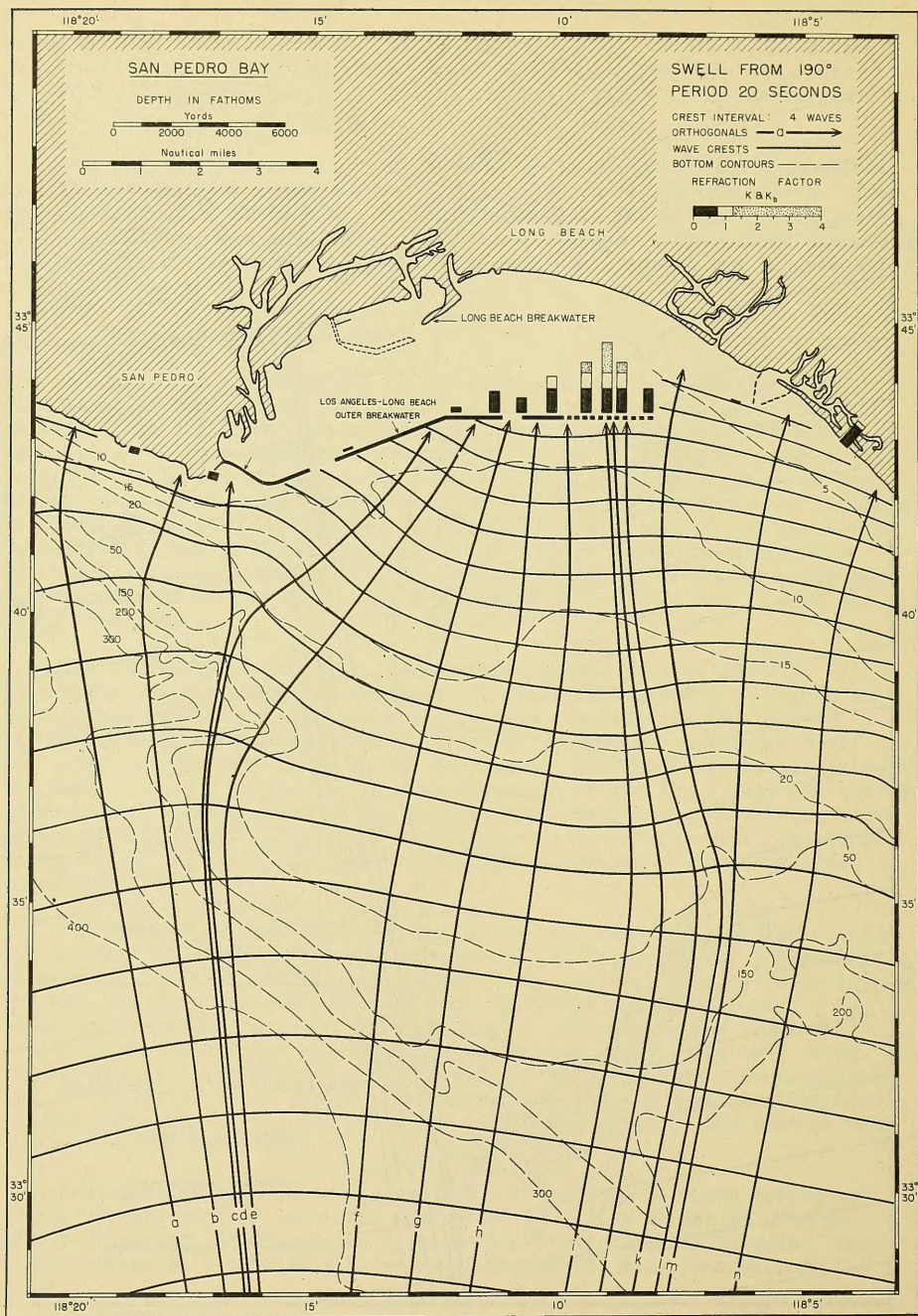


FIGURE 3

submarine valley located south of San Pedro, and leads to a maximum divergence between orthogonals c and d. Due to the focusing of wave energy by San Pedro Shelf, the wave crests between orthogonals g and k converge on the eastern portion of the breakwater, the greatest convergence occurring between i and j. On both sides of the breakwater the refraction factors indicate divergence.

Figures 4 and 5 show the detailed refraction diagrams for southern hemisphere swell advancing toward San Pedro Bay from 180° and 170° respectively. Although the main features of these figures are similar to those of Figure 3, several significant differences exist. These variations are shown best in Figure 6 which gives the refraction factors along the breakwater for all three diagrams. In addition, the refraction factors are plotted for 20 second waves from 160° as computed from the diagram presented by O'Brien (1947).

Figure 6 shows that an eastward shift in the direction of wave approach causes the zone of convergence to shift westward along the breakwater. The westward shift of the general pattern of wave refraction for 160° beyond that of 170° serves to support this. In the region of greatest divergence along the western portion of the breakwater greater intensity of wave energy is experienced from 160° than from the other three directions because the shadowing effect of the islands is lacking for 160° . In this connection the combined effects of the islands and San Pedro Shelf in shifting the entire refraction pattern may be compared to the action of a fulcrum, about which the wave approach and wave advance are pivoted.

The region of convergence indicated for directions of wave approach from 160° to 170° coincides closely with the location of the damaged portion of the Long Beach Breakwater in 1930. Applying a refraction factor of 3.50 for this zone of convergence and utilizing the available observations¹ of wave height, which were made during this critical period at locations seaward of the outer end of the breakwater, a deep water wave height of between 3 and 4 feet is obtained.

For southern hemisphere swell of 20 second period the refraction factor would be somewhat modified for slightly different period swell; shorter wave periods would be accompanied by less convergence.

Part II - Waves from a Tropical Storm.

The large waves at San Pedro Bay on September 24 and 25, 1939, were produced by unusually strong winds in the area of the North Pacific to the south of Long Beach. Unlike southern hemisphere swell which originates beyond the present weather reporting area, these waves could be estimated in advance by using available synoptic weather data. In this study the wave characteristics at San Pedro Bay have been computed for September 24 and 25 by use of the forecasting technique given by Francis (1944) and a set of wave forecasting graphs prepared by Arthur (1948) from the revised theory given by Sverdrup and Munk (1947).

Investigation of synoptic weather situation. - When the tropical storm appeared on the weather maps it was progressing northward along the

¹All footnotes are given at the end of the article.

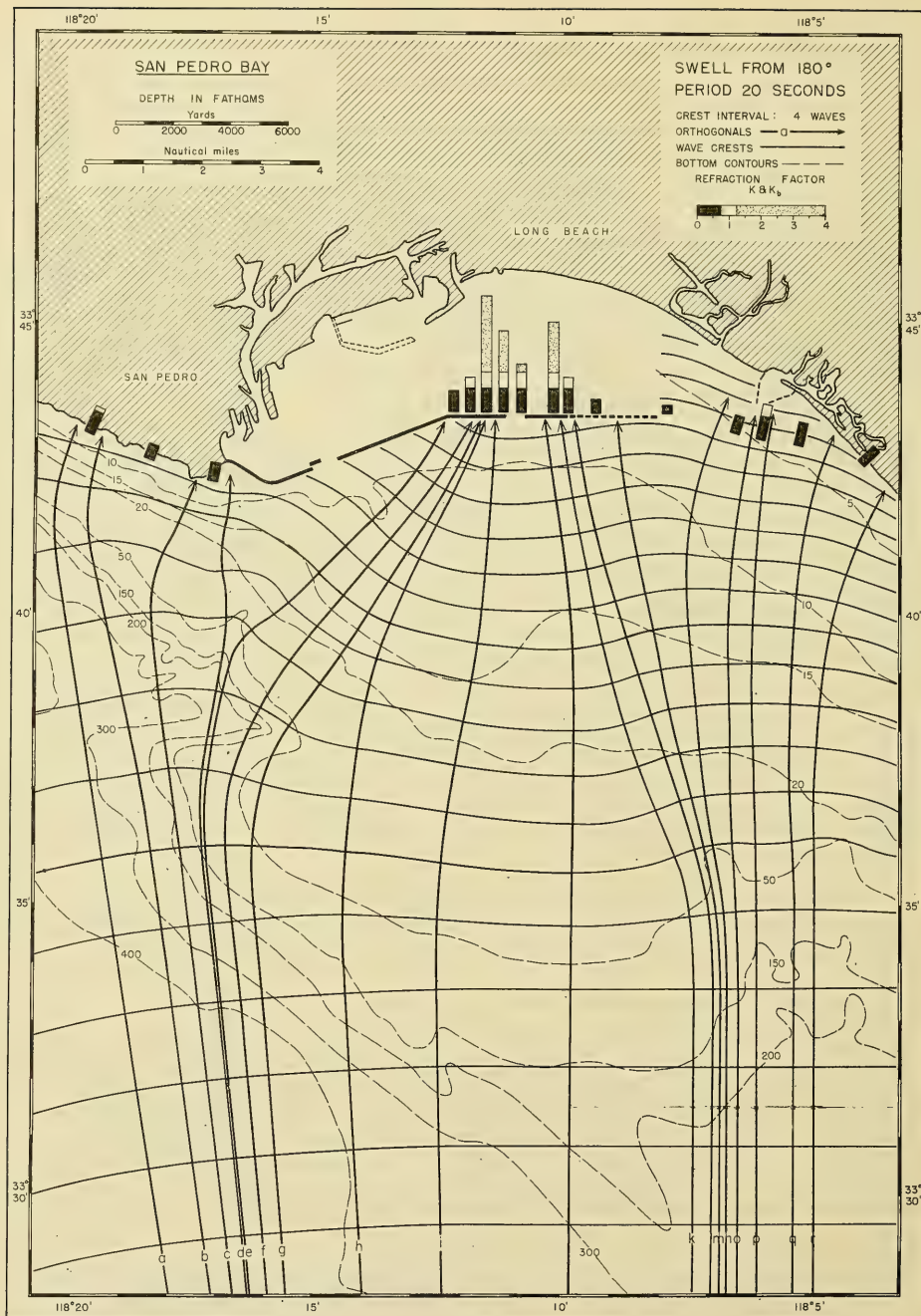


FIGURE 4
6

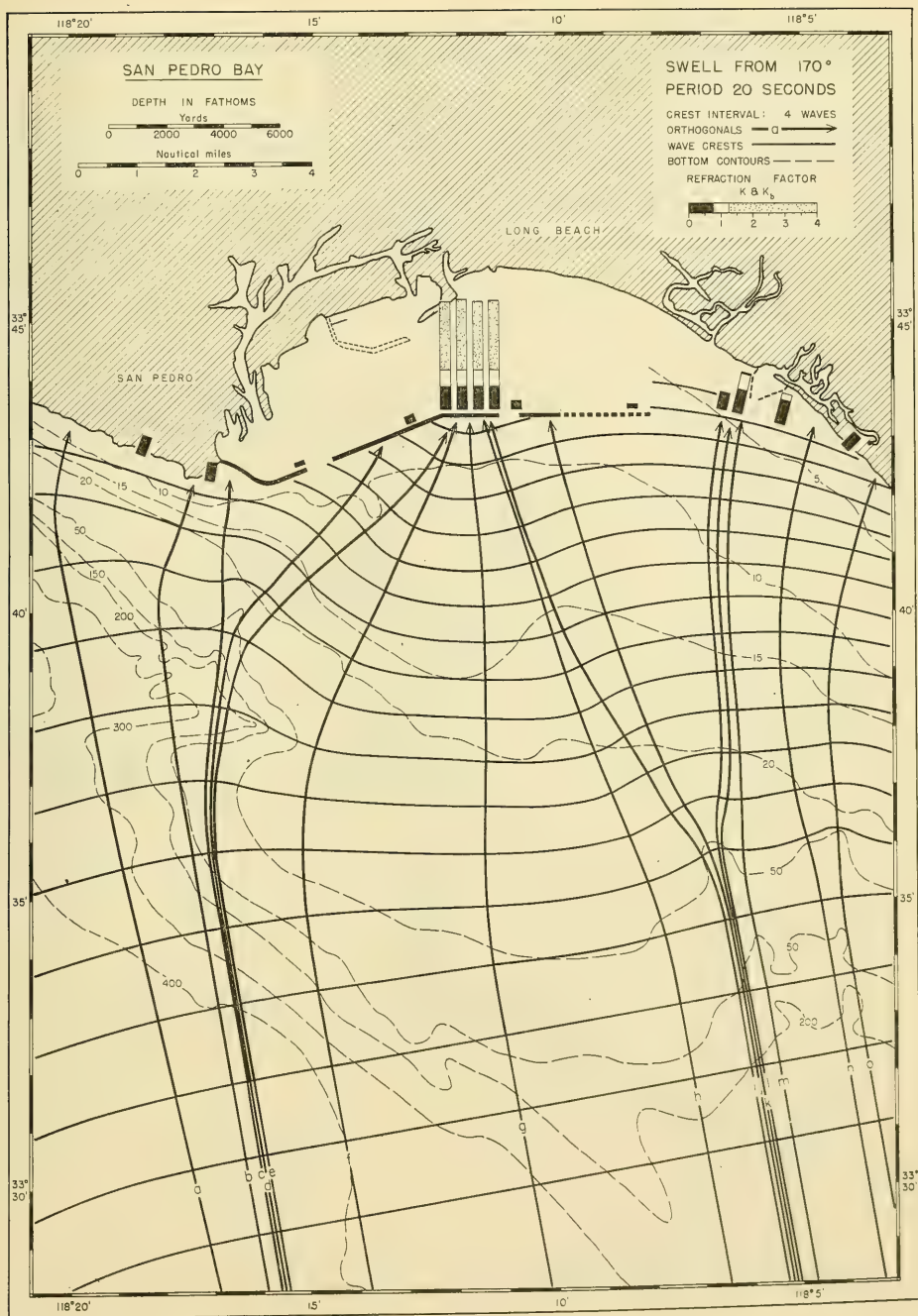


FIGURE 5

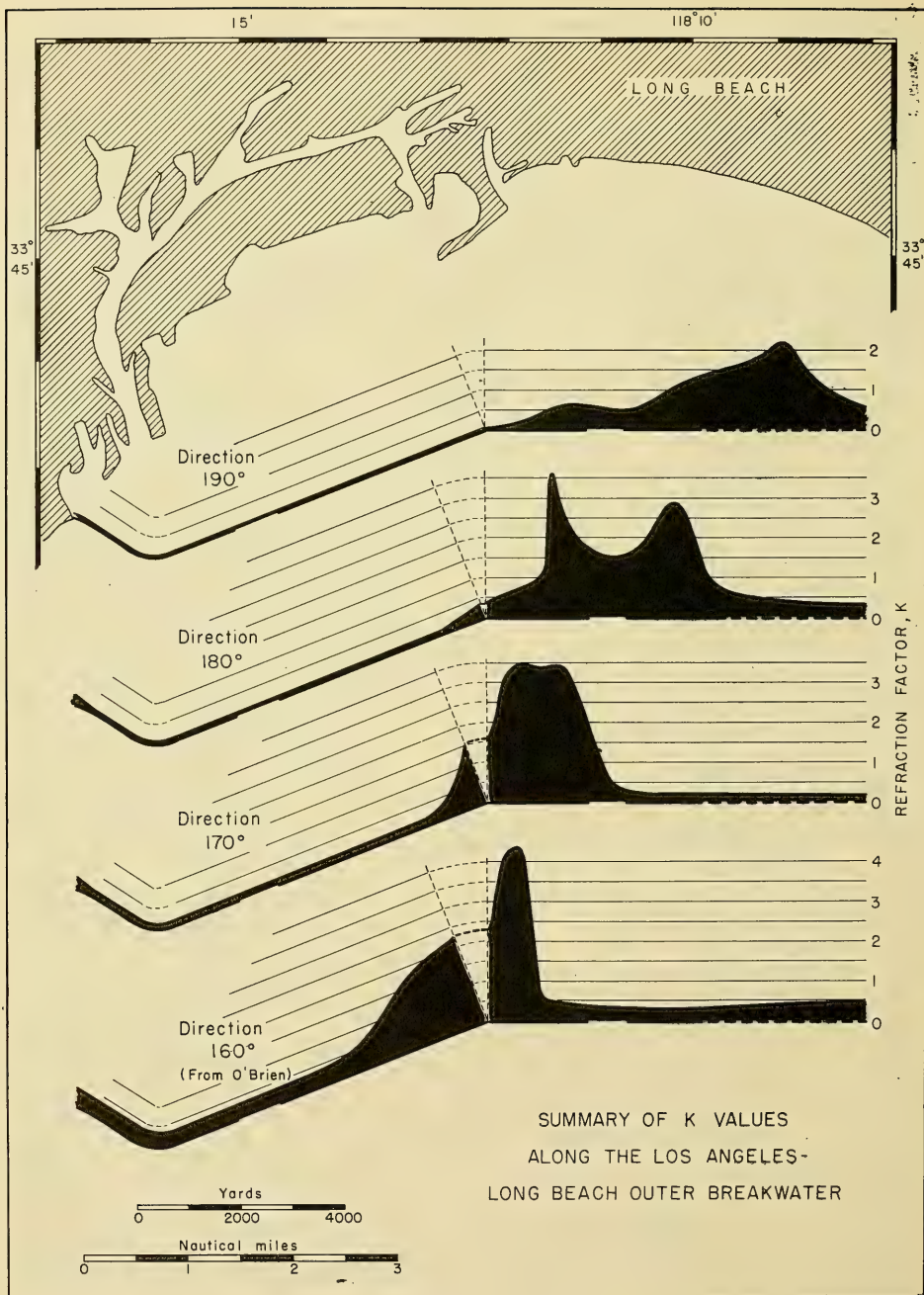


FIGURE 6

Lower California coast, and it subsequently dissipated after it encountered the coast of Southern California. The weather maps in Figure 7 show that the storm decreased in intensity from September 22 to September 25. As it traveled northward toward the coastline the winds in the eastern portion of the storm were always directed toward Long Beach. This condition, together with the movement of the storm, produced an effective fetch which exceeded 1,000 miles. The velocity of the storm's center was approximately 15 knots during the interval 1630 P.S.T., September 21 to 1630 P.S.T., September 22, after which it gradually decreased to about 6 knots by 1630 P.S.T., September 24.

In order to determine the longest possible effective fetch, within the limits of available weather data, the speed of the wave energy front which remained under the influence of strong winds for the longest time was computed relative to the speed of the storm. This energy front would be expected to emerge from the fetch before the strong winds from the storm reached San Pedro Bay. However, because the wind field quickly elongated and was extended to Long Beach during the period 0430-1630 P.S.T. September 24 as shown in Figure 7, the waves comprising the energy front continued to grow under these winds all the way to Long Beach. In addition, the extended wind field represented an earlier local fetch which produced short period wind waves of considerable height.

Between 1630 P.S.T., September 24 and 0430 P.S.T., September 25, the storm elongated in an east-west direction. Soon after the 0430 map the winds at San Pedro Bay subsided to calm or light and variable, and the storm center moved toward the coast in the region between San Pedro and San Diego. At about 0700 the winds at San Pedro backed to the northeast, but a small fetch still existed to the south. The existence of this fetch is verified by the south winds which were recorded at San Pedro for some period after the windshift occurred at San Pedro (Table 1). By 1630 P.S.T., September 25 the storm's center was no longer distinguishable.

Computed deep water wave conditions. - The computed deep water characteristics at San Pedro Bay for waves from the tropical storm are tabulated in Table 2 and presented graphically in Figure 8. Swells from westerly directions preceding and following the waves from the tropical storm are also shown on the graph. The time when generation began as shown in Table 2 was the initial time at which the average wind velocity, $\bar{U}$, applied. Prior to this time lighter winds which preceded the main fetch had produced low waves of short period. These waves were still present in the fetch area at the time generation began. As a result, the actual fetch, $\bar{F}$, shown on the weather maps was effectively increased by the distance required to generate the short low waves at velocity, $\bar{U}$. This distance together with the fetch shown on the weather maps and the distance the storm moved with the waves during generation comprise the effective fetch, F_e . The wave height and period at the end of the fetch, H_F and T_F , have been computed using either $\bar{F}$ or F_e , whichever was greater.

The opposing wind which existed in the decay area after about 0700 P.S.T. September 25 was allowed for by increasing the decay distance, $\bar{D}$, by a certain amount (Arthur, 1948). The wave height and period at the

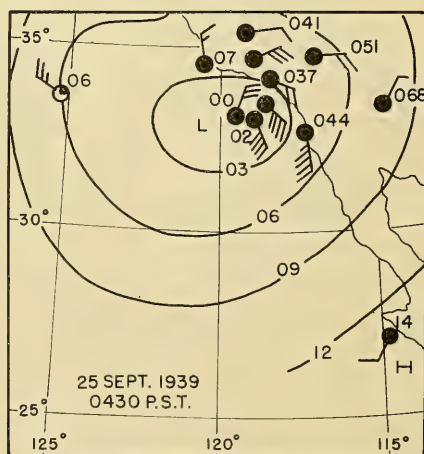
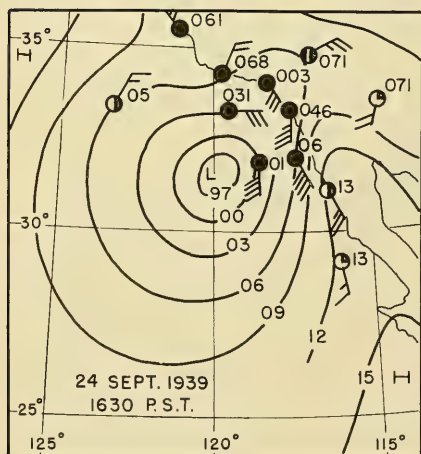
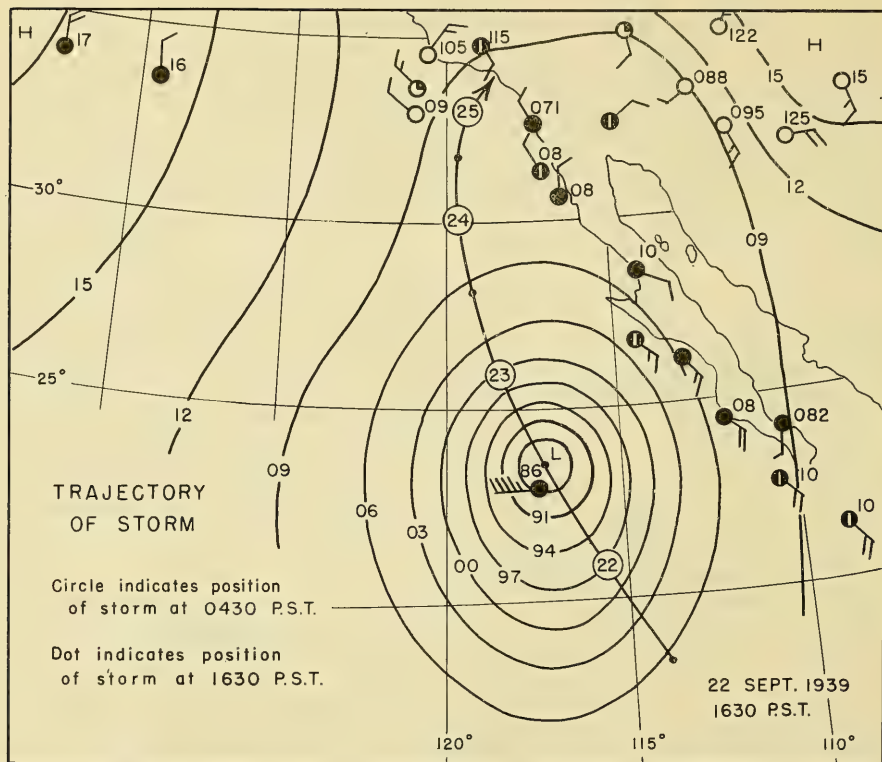


FIGURE 7
10

TABLE 1

Hourly Winds for Coastal Stations in Southern
California during the Tropical Storm of September 1939

<u>LOS ANGELES AREA</u>				
Date and Time	<u>SAN DIEGO</u>		<u>LOS ANGELES**</u>	<u>SAN PEDRO</u>
	Direction	Velocity in MPH	Direction	Velocity in MPH
24-0900*	S	7	SE	3
1000	S	12	SE	3
1100	SE	24	SE	9
1200	S	29	SE	21
1300	S	29	SE	30
1400	SE	30	SE	30
1500	SE	25	SE	40
1600	SE	27	SE	42
1700	SE	26	SE	36
1800	SE	25	SE	34
1900	SE	24	SE	40
2000	SE	24	SE	37
2100	SE	25	ESE	39
2200	SE	24	ESE	39
2300	SE	24	ESE	35
25-0000	SE	24	ESE	32
0100	SE	22	ESE	40
0200	SE	24	ESE	38
0300	SE	24	ESE	41
0400	SE	29	ESE	43
0500	SE	27	ESE	41
0600	S	29	ESE	34
0700	S	24	NE	26
0800	S	22	NE	16
0900	S	21	NW	12
1000	S	19	NW	15
1100	S	18	W	18
1200	S	15	W	24
1300	S	15	W	22
1400	S	15		
1500	S	13		

* September 24, 1939, 0900 Pacific Standard Time.

** Wind directions were not recorded at San Pedro.

TABLE 2. - Computations of Deep Water Wave Conditions for San Pedro Bay
during Tropical Storm of September, 1939

Time and date genera- tion began at wind velocity, U	0930 9-24	1000 9-24	1030 9-24	1100 9-24	1130 9-24	1130 9-21	2000 9-21	0700 9-22	1730 9-22	0430 9-23	1030 9-23	1630 9-23	2230 9-23	0430 9-24	1030 9-24	1630 9-24	2230 9-24	0430 9-25	Small waves existing in rear portion of fetch
Average wind velocity, U (knots)	15	20	30	35	37		36	36	36	36	36	36	36	36	37	37	38	39	
Fetch, F (naut. miles)	165	165	160	155	150		200	200	200	200	195	190	185	170	165	145	85	35	
Effective fetch, F _e (naut. miles)																			
Duration of wind, t _d (hours)	1	2½	3	4½	8														
Minimum duration, t _{min} (hours)	28	24½	20	18	17		83	75	67	59	51	46	42	36½	31	25½	20	14	8
Wave height at end of fetch, H _f (feet)	2.0	3.4	6.5	10.4	16.8		28.5	28.2	27.9	27.6	27.3	26.9	26.7	26.4	26.0	25.0	24.0	23.0	18.0
Wave period at end of fetch, T _f (seconds)	2.0	2.5	3.6	4.6	5.9		14.0	13.4	12.7	12.4	11.7	11.3	10.8	10.3	9.8	9.2	8.4	7.5	6.1
Time when waves leave fetch and enter decay area												0530 9-25	0730 9-25	0800 9-25	0830 9-25	0900 9-25	0930 9-25	0930 9-25	1130 9-25
Decay distance, D (naut. miles)												15	20	30	45	60	70	70	70
Wind velocity in decay area, * U' (knots)												-18	-18	-18	-18	-18	-20	-20	-15
Effective decay dist., D _e (naut. miles)												31	47	72	98	118	130	142	134
Height at end of decay area, H _p (feet)												26.8	25.8	24.8	23.4	21.2	19.0	17.0	11.5
Period at end of decay area, T _p (seconds)												11.3	10.9	10.5	10.1	9.8	9.2	8.5	7.3
Travel time to end of decay area, t _p (hours)												1	1¼	2	3	4	5¼	5 ¾	8 ¾
Computed arrival time	1030 9-24	1130 9-24	1230 9-24	1330 9-24	1430 9-24	1730 9-24	2000 9-24	2300 9-24	0130 9-25	0430 9-25	0630 9-25	0845 9-25	1000 9-25	1130 9-25	1300 9-25	1445 9-25	1515 9-25	1730 9-25	2115 9-25

*Negative sign indicates direction of wind was opposite direction of wave movement

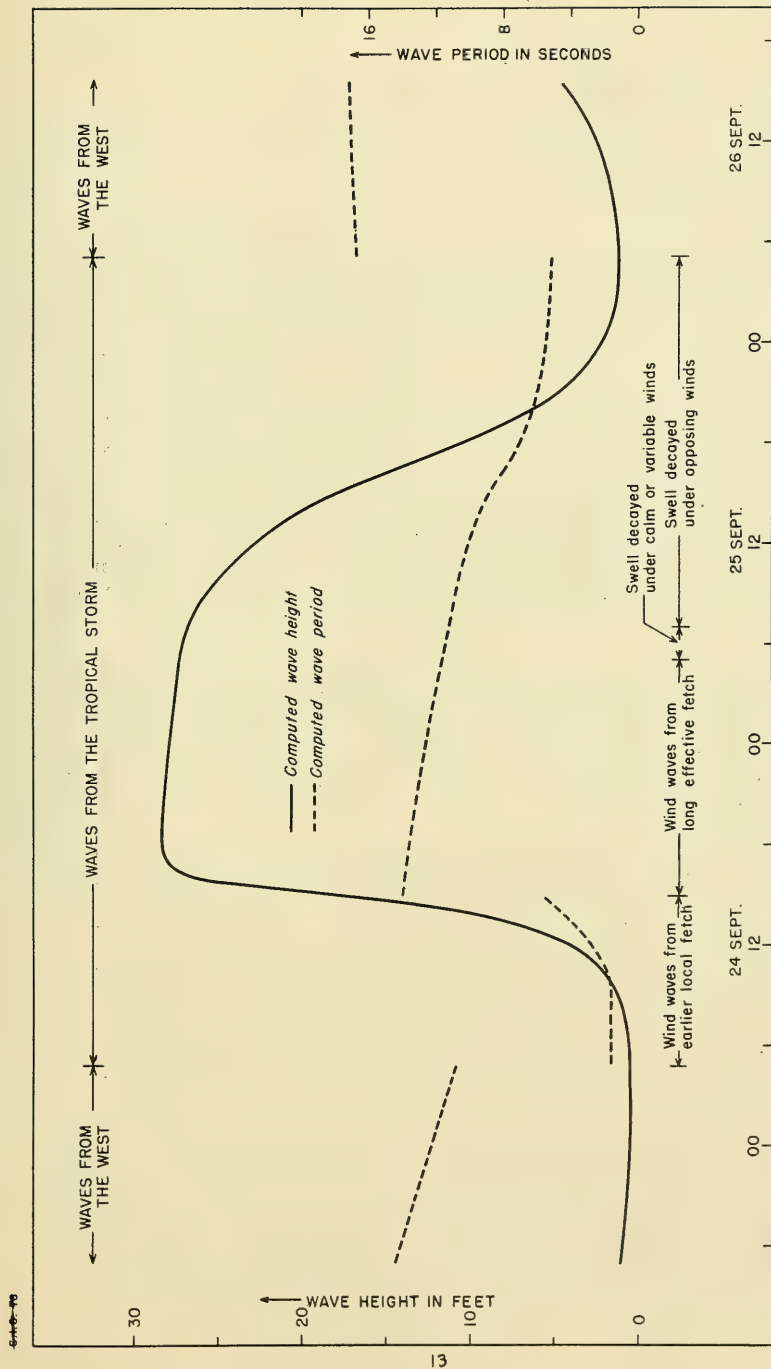


FIGURE 8 - COMPUTED DEEP WATER WAVE CONDITIONS AT SAN PEDRO BAY

end of the decay area, H_D and T_D , which are shown in Table 2, have been computed using the effective decay distance, D_e , where applicable. In the computations no diffraction correction (Francis, 1944) has been applied because (1) no decay distance existed until after 0430 P.S.T., September 25 and (2) after this time the fetch no longer resembled the circular fetch characteristic of a tropical storm.

The waves from the tropical storm may be divided into four parts as shown in the lower part of Figure 8. The short period wind waves from the earlier local fetch increased rapidly in height at about 1200 P.S.T., September 24 and remained predominant until about 1430 P.S.T., September 24. At that time waves from the long effective fetch arrived. These had not traveled through decay areas and were still growing under the influence of strong winds when arriving at San Pedro Bay. They were of great height and relatively long period. The rapid build-up in wave height shown in Figure 8 would have been even more accentuated if the earlier local fetch had not developed. Such a quick build-up of wave height is a characteristic common to waves from tropical storms, and it has been reported for certain other tropical storms that strong winds and very high waves arrived simultaneously. Soon after the arrival of waves from the long effective fetch the computed wave height was greater than 28 feet and the corresponding period 14 seconds. For almost 12 hours these waves maintained great height, but their period steadily decreased with time.

The waves shown between 0500 and 0700 September 25 represent swell from the small fetch to the south decayed under calm or variable winds. After 0700 when the winds at San Pedro backed to northeast the waves represented swell decayed under opposing winds, and the height and period decrease fairly rapidly. Not until about 0500 September 26 did the swell from the west become larger than the swell from the weak residual fetch located to the south.

Refraction pattern and distributing of damage. - As indicated by the geographical locations of the tropical storm's wind field shown in Figure 7 the wind waves and swell from this storm approached Long Beach from directions varying between 150° and 190° in deep water. Figure 9 is a portion of the refraction diagram for 175° and 14 seconds period. This corresponds to the period of the highest waves from the long effective fetch reaching San Pedro Bay. The complete diagram from which Figure 9 is taken is shown in the inset and was drawn with a crest interval of 2 waves on a detailed chart of the bottom topography. The refraction factors are presented in the same manner as on the diagrams for southern hemisphere swell.

The damaged portions of the Los Angeles-Long Beach Detached Breakwater and the intensity of the damage within these areas are also shown in Figure 9. It is evident that the amount of damage was increasingly great from west to east along this breakwater while the Long Beach breakwater escaped without damage. From this it appears that the wave forces were considerably greater near the easterly end of the detached breakwater. Because of its seaward location, the detached breakwater apparently provided shelter for the undamaged Long Beach Breakwater.

175° was about the most easterly direction from which waves could have produced the observed damage pattern. From more easterly directions the waves probably would have resulted in damage to the Long Beach Breakwater. Consequently, it appears that the damaging waves approached San Pedro Bay from directions within the narrow sector from 175° to about 185°. For directions west of 185° the effect of refraction around Catalina Island considerably reduced the wave force on the breakwater.

Figure 9 shows that the refraction factors increase progressively toward the east end of the detached breakwater from 0.37 between orthogonals a and b to 1.65 between orthogonals f and g with corresponding heights varying from 10 to about 45 feet, disregarding interference between wave trains.

Unfortunately, actual measurements of wave height and period at San Pedro Bay during this storm were not made², but the direction of wave approach was observed by means of a transit from Pt. Fermin near the westerly end of San Pedro Breakwater. This observation indicated that waves came from the southwest concurrently with waves from the southeast, and that the southwest waves appeared to be the larger. Although, it is impossible to determine without observations of wave period whether or not one train of waves was southern swell arriving simultaneously with the waves from the tropical storm, the observed directions of wave approach and distribution of damage can be fully explained by the tropical storm waves alone.

Since waves from a tropical storm are propagated radially from the storm's center, there is a certain amount of variability in direction from which the waves may come. The higher waves arriving at San Pedro Bay were probably generated near the center of the storm, but at the same time some waves were being generated by the weaker winds nearer the east side of the storm just off the coast, and these smaller waves approached Long Beach from directions which were more to the southeast. Having less height and shorter periods they experienced little refraction in crossing San Pedro Shelf and reached the breakwater retaining their original direction. On the other hand, the large waves from the long effective fetch approaching from directions between 175° and 185° were refracted most over San Pedro Sea Valley so that the wave crests in the vicinity of the breakwater were oriented in such a way that the waves appeared to come from directions more toward the southeast.

Summary and Conclusions

The characteristics of waves destructive to the harbor breakwaters in the Long Beach-San Pedro area have been examined. As a result of refraction over the complicated bottom topography in San Pedro Bay this wave energy was concentrated at certain parts of the breakwaters. The refraction effect of the bottom topography upon waves from the tropical storm was similar to that for southern hemisphere swell but less pronounced because the tropical storm waves had shorter periods.

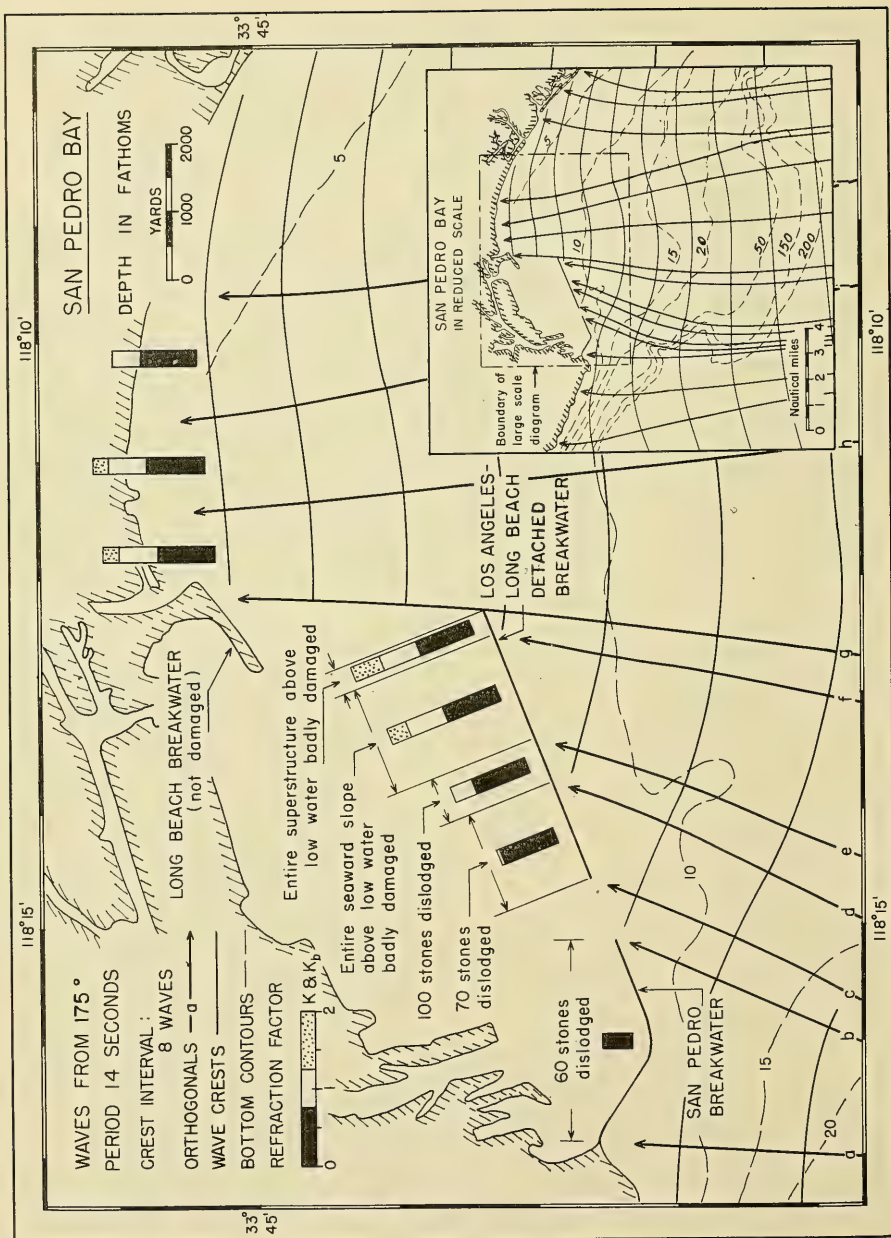


FIGURE 9

The predominant characteristics common to all the preceding refraction diagrams can be attributed to specific features of the bottom topography. In each case a zone of divergence is produced by the sea valleys located on either side of the San Pedro Shelf. And, these valleys aid in producing greater convergence over the extended shelf lying between them. Slight changes in wave direction result in sizeable displacement of the zones of convergence and divergence, while the effect of wave direction upon the magnitude of the refraction factor is somewhat less marked. Because of interference between wave trains, the largest waves at certain parts of the breakwater may contain greater intensity of energy than the amount indicated by the refraction factors.

The computed wave height and direction of approach of the tropical storm waves agree with observed conditions. Without additional observations of both wave height and period, it is impossible to test the accuracy of the existing technique for estimating waves from a tropical storm. The computations indicate that waves at Long Beach from such a tropical storm contain far greater energy than wind waves or swell from other northern hemisphere storms which pass to the south of Long Beach. However, because of the rarity of occurrence of tropical storms which can produce damaging waves in the Long Beach region it may be more economical and practical to plan to repair the existing breakwaters after each of these storms than to build the breakwaters heavy enough to withstand such wave attack.

Acknowledgements

The U. S. Weather Bureau, Department of Commerce, and the California Institute of Technology have furnished the meteorological data needed for this study. Mr. R. O. Eaton, Corps of Engineers, has provided information pertaining to wave observations and damage to harbor structures in the Long Beach-San Pedro area.

Footnotes

- 1 Measurements of wave height were made by Mr. R. O. Eaton, Corps of Engineers and communicated by letter to Dr. Walter H. Munk of Scripps Institution of Oceanography. Wave heights of 15 and 18 feet were recorded in depths of 37 and 27 feet, respectively.
- 2 Lieut. H. F. Nowak, U.S.N., has recently given a verbal description of the height of the wave crests as he observed it from aboard the aircraft carrier Lexington on September 24, 1939. The Lexington was at anchor in water between 50 and 60 feet deep east-northeast of the eastern end of the detached breakwater. Lieut. Nowak estimated the tops of the wave crests to be about 24 feet above mean water level. If it is assumed that the trough of the waves is one-fourth of the wave height below the mean water level the total wave height is 32 feet. The computed wave height in this area, allowing for refraction and depth, is between 32 and 39 feet, depending upon the exact position for which the refraction factor is computed.

REFERENCES

- Arthur, R. S., 1948. Revised Wave Forecasting Graphs and Procedure, Wave Project Number 73, Scripps Institution of Oceanography of the University of California, 14 pp.
- Francis, W. J. Jr., 1944. Waves and Swell from a Tropical Storm, Wave Project Report No. 29, Scripps Institution of Oceanography of the University of California, 29 pp.
- Munk, W. H. and M.E. Traylor, 1947. Refraction of Ocean Waves: A Process Linking Underwater Topography to Beach Erosion, Journal of Geology, Vol. LV, No. 1, pp. 1-26.
- O'Brien, M. P., 1947. Wave Refraction at Long Beach and Santa Barbara, California, Technical Report HE-116-246, Fluid Mechanics Laboratory Berkeley, California, 14 pp.
- Sverdrup, H. U., and W. H. Munk, 1947. Wind, Sea and Swell; Theory of Relations for Forecasting, U. S. Hydrographic Office, Technical Report No. 1, H. O. Publication No. 601, 44 pp.

THE WIND ELEMENT IN BEACH EROSION

By

Martin A. Mason

Chief, Engineering and Research Branch

Beach Erosion Board

(Presented at Symposium on Hydrometeorological Problems, American Geophysical Union, 2 May 1950)

The art of beach protection against loss of sand has been developing for decades through the thought and experiment of engineers, geologists, and others whose interest derived chiefly from circumstance and only rarely for other reasons. Early students of shore control problems had little to guide their thinking and experiment other than their observational acumen. Thus, waves were recognized as a factor causing transport and possible loss of sand, but local wind, usually most evident when waves were observed, was considered to be of prime importance. Possibly much of the regard in which local wind has been held as an important factor in beach erosion stems from the early reports of the Beach Erosion Board. These reports accorded a prominent place to discussion of the local wind and its probable effects. Today, when it is believed that the mechanism of loss of beach is better understood than previously, there is a notable tendency to assign to local wind a much less important place as a factor in sand loss.

Wind is believed presently to be the ultimate source of the energy which, when expended on a beach, either directly as wind or indirectly through the medium of surface water waves, causes transportation of sand along the shore and the bottom, and ravages the land boundaries of the water mass. In the process of expending energy on the shore local winds are recognized now as playing usually a relatively minor role.

Investigators long have recognized the beach as the primary source of supply for shore dunes and correctly identified local wind as the eroding and transporting agent responsible for the dune formations. The conditions requisite to erosion, or deflation of beaches by local winds can be stated as:

1. An expanse of dry beach sand, of sufficient extent in the direction of wind movement and so located with respect to the surrounding topography as to permit sand transport by a local wind system.
2. The existence of wind in intimate proximity to the sand surface.

This second condition appears at first glance to be trivial, however, those familiar with beaches located adjacent to appreciable topographic irregularities often have experienced absence of wind immediately contiguous to the sand surface when local wind was present elsewhere.

So far as is known presently the mechanism of erosion and transport of sand by wind, is analogous in all respects to that by water; the Reynolds numbers of the process in the two media is similar, and observations of the actual movement of sand grains shows the motion to be analogous in air and water. An additional factor not yet investigated to the author's knowledge is that of the role played by water content of the beach in causing a time delay in the erosion-transportation sequence. The differences in character of a given sand or beach mixture when dry or wet have been noted often; generally, moist sand has a higher load bearing capacity than dry, while wet sand often shows a much lower capacity than dry. Moist sand appears to exhibit more cohesion than the dry, while wet sand appears to be more fluid even than the dry material. Yet the author has observed apparently dry sand blown in considerable quantity from beach areas still wet from a receding tide. O'Brien and Rindlaub⁽¹⁾ report that "on one occasion, sand was unpleasantly evident at 8 feet above the beach during a heavy rainstorm accompanied by wind with an intensity of 48 miles per hour." They report also an observation that wind of 15 miles per hour velocity moved sand in the moist area below the last high water mark. The evaporative effects of the wind seem to be important to the problem, probably entering chiefly by changing the erodibility characteristics of the material.

Bagnold⁽²⁾ has shown that sand movement in air occurs as a sand cloud (sand in suspension) and as surface creep (bedload), in analogy to sand movement by water. Similarly to sand movement in water the motion of the sand grains occurs as rolling, saltation, and suspension. His experiments show that the threshold velocity required to sustain sand movement can be expressed as:

$$V_t = 0.47 \left(\frac{\sigma - \rho}{\rho} g d \right)^{1/2} \log \frac{30k'}{d}$$

where σ and ρ are the densities of the sand grain and air respectively; d is the grain diameter; and k' is the height above the surface at which the threshold velocity occurs, related to the surface roughness (parameter K in the Prandtl turbulent flow law).

Through application of Prandtl's rough-surface law for turbulent flow and consideration of surface drag relations Bagnold suggests the following expression for relating total sand flow to wind velocity

$$Q = C \sqrt{\frac{d}{D}} \frac{\rho}{g} \frac{(V_z - V_t)^3}{5.75 \log^2 k'}$$

where d is the mean diameter of any sand; D the mean diameter of a so-called standard sand (0.024cm); V_z the wind velocity measured at a height Z above the sand surface; k' has the value 0.3 cm. for uniform sands; and C is a coefficient varying from 1.5 for uniform grading of sand to 2.8 for sand with a wide range of grain size.

- (1) The Transportation of Sand by Wind - M. P. O'Brien and B. D. Rindlaub, Civil Engineering, Vol. 6, No. 5, May 1936.
- (2) The Transport of Sand by Wind, R. A. Bagnold, The Geographical Journal Vol. LXXXIX, No. 5, May 1937.

Shore dunes are the visual evidence of a sedimentation process and occur as a result of a local climatic condition by which the sand transporting wind loses its competence. This may occur by reduction in wind velocity; by change in wind direction; or by sheltering, either natural or induced artificially.

So long as sand is available on the beach and local wind conditions favorable to sand transport exist the process of deflation continues. O'Brien and Rindlaub⁽¹⁾ report rates of movement as high as 2,200 pounds of sand per foot of beach per day measured on Clatrop Spit at the mouth of the Columbia River. In many areas the loss of sand from beaches by wind deflation is doubly troublesome, as at Hollywood Beach near Port Hueneme where dune formations induced by shore cottages threatens to engulf the buildings, the structures acting as effective sand fences in promoting dune formation and growth. In this area the loss of sand by local wind action probably approximates 50,000 cubic yards annually, which is more than sufficient to maintain excellent beaches almost anywhere in the Great Lakes area where annual littoral drift is smaller than on most ocean shores.

The indirect effects of wind on the shore face are much more important than the direct effects briefly discussed above. The case may be stated in this fashion. If wind is assumed to blow over an initially rough water surface energy will be transferred from the wind to the water by normal pressure and frictional drag, thus generating wave motion. The area in which this transfer occurs, i.e., the generating area, corresponds roughly to the area of the wind system. Considering the wide extent of the relatively unobstructed water surface, and the possibility in large bodies of water that several wind systems may contribute to the energy of waves, it can be presumed that the energy carried to the shore by waves greatly exceeds that which it is possible to obtain by the action of local winds.

The energy budget of wind generated waves has been studied by Sverdrup and Munk⁽³⁾, who have stated that "It seems probable that the energy used for wave formation represents only a small fraction of the wind energy which is dissipated below a height of 8 to 10 meters.---- A very small fraction, about 1 per cent, of this amount is transferred to the sea for maintaining the pure wind current and another fraction goes into formation and maintenance of waves. ---- Analysis indicates that 10 per cent or less of the wind energy dissipated in the lowest layer (of wind) goes toward increasing or maintaining the energy of the waves." Among other factors to be considered are: that,

1. The waves existing at any moment are the result of wind action in one or more generating areas;
2. Interference occurs between waves;
3. The generating wind is variable in velocity, direction, and duration;
4. A wind of given velocity probably generates a spectrum of waves of various periods and amplitudes.

(3) Wind Waves and Swell-A Basic Theory for Forecasting, H. U. Sverdrup & W. H. Munk, Scripps Inst. of Oceanography, Wave Rpt. No. 1, Sep 1943.

Sverdrup and Munk succeeded in establishing a fundamental equation relating wind action to wave action, and showing that the energy content of the waves generated depends primarily upon the wind velocity, the wind fetch, and the wind duration.

Waves forecast by the Sverdrup-Munk relationship have been shown by comparative observation to approximate reasonably well natural conditions, thus confirming their theoretical concepts. It can be concluded that the waves reaching a shore are, in most instances, due to the net effect of one or more wind systems transferring energy to the water, with the wave motion serving as a collecting and transporting medium conveying the net energy contained in the waves to the shore.

At the shore interface wave motion can no longer exist and the wave energy built up over long intervals of time and space must be dissipated abruptly. A large part of the energy is believed to be lost in the turbulence of the breaking wave, in the pick-up and throwing into suspension of bottom or beach material, in impact, and other losses, the remainder being lost in the rush of the broken wave, carrying its load of suspended sand, up the beach slope. This is an ideal situation for the erosion and transport of large quantities of sand or beach material. Waves commonly approach the shoreface at an angle and there is a resulting component of motion of sand and water along the shore, giving rise to the well-known littoral transport of sand.

The annual rate of loss of sand by littoral transport due to wind-generated waves varies widely in different situations, the maximum known to the author occurring south of Port Hueneme, California, and approximating 1,000,000 cubic yards. This rate is some twenty times the estimated loss due to local wind action in the same general area. In other areas, such as the south Florida coast losses due to local wind are not discernible while littoral transport approximates 250,000 to 270,000 cubic yards annually.

The effects of wind action on water in causing wind-generated currents and wind set-up or depression are considered to be negligible in the usual shore problem in comparison with the effects discussed above.

It should not be presumed that the effect of wind on beaches is wholly destructive. In fact the destructive effects of local winds in denuding beach areas beyond the range of normal wave and tide action can be turned to good purpose by management of the wind-transported sand. The use of planting and sand fences to build dunes on or immediately adjacent to the landward portions of a beach is an effective means of conserving and stockpiling sand against the day of unusually destructive storm wave or tide action. This conservation practice has been notably successful in rebuilding and maintaining the protective dune ridge of the south shore of Long Island. In the September 1938 hurricane that devastated much of the New York and New England area it was the dune ridge built naturally by local wind action and acting as a dike that prevented engulfment of the Great Barrier beach of Long Island by the sea. Further, the sand of the dunes, once lost from the beach, was returned to the beaches by the waves to preserve the famous strand characteristic of this area. Careful management

of the local wind transported sand since that time has rebuilt the dunes and reestablished the natural stockpile of sand against the next hurricane.

The wind element in beach erosion must be considered as the one most important factor of the entire problem. Its net influence appears to be degenerative, and not susceptible of effective control. It is most apparent in its indirect effect of wave generation, however, in many instances its direct effects appear to be subject to management in a conservative fashion.

* * * * *

THE LAG AND REDUCTION OF RANGE IN TIDE GAGE WELLS

The following paper first appeared in limited issue as Technical Memorandum No. 14, U. S. Tidal Laboratory, Berkeley, California, by Morrough P. O'Brien, Consulting Engineer. The paper is reproduced here to bring the findings to the attention of a larger group of research workers and others having an interest in tidal phenomena.

Abstract

The problem of the lag of high and low water and the reduction of range has been treated by Chapman (Philosophical Magazine, Vol. 46, 1923), for the case of a simple orifice connecting a primary tidal basin with a secondary basin on the assumption that the instantaneous rate of flow corresponds to the instantaneous head. In the analysis which follows, the treatment is generalized to include the effect of both laminar and turbulent friction and the effect of acceleration.

Theory

General Equations. - The equation of continuity relating the average velocity in the connecting line and the velocity of rise of the water surface in the well is (for symbol meanings see Appendix I)

$$aV = A \frac{dH'}{dt} \quad (1)$$

At any instant, the elevation of the primary tidal surface is H and that in the gage well is H' . The working head at that instant is

$$h = H - H' \quad (2)$$

Flow occurs into the well when $H > H'$, or $h > 0$, and out of the well when $H < H'$. The primary tidal surface exhibits periodic variations and for the purpose of this analysis, these variations will be assumed to be sinusoidal, i.e.

$$H = H_0 \sin \alpha t; \quad \alpha = \frac{2\pi}{T} \quad (3)$$

The curve of H' will also be a sine curve of lesser amplitude lagging the curve of H but having the same period. Therefore, the equation for the working head may be written as

$$\begin{aligned} h &= H_0 \sin \alpha t - H_0' \sin \alpha (t + \epsilon) \\ h &= h_0 \sin \alpha (t + \epsilon) \end{aligned} \quad (4)$$

Here, ϵ is the time interval by which corresponding tidal events in the tide gage well lag those in the outside basin. The equation of continuity may also be written:

$$aV = A \frac{d(H-h)}{dt} \quad (1a)$$

Laminar Flow. - For the usual tide gage installation in a tidal model, the flow in the connecting pipe line is probably laminar. The criterion for the occurrence of laminar flow is that

$$\frac{VD}{\nu} < 2300 \text{ (approx.)} \quad (5)$$

Since the velocity increases periodically from zero to a maximum, V_m , the Reynolds number (VD) increases from zero to a maximum value, R_m , and for the purpose of limiting the present treatment, it will be assumed that for laminar flow

$$R_m = \frac{V_m D}{\nu} < 2300 \quad (5a)$$

Periodic reversal of flow introduces some disturbance and the critical value of the Reynolds number may be slightly lower than 2300 but there are no data available on the magnitude of the reduction.

Poiseuille's equation for laminar flow in a long tube is

$$V = \frac{wR^2 h}{8\mu L} \quad (6)$$

The tube is assumed to be long in order that the velocity head loss at discharge will be negligible as compared with the overall working head, h . The head necessary to accelerate the water in the connecting pipe is neglected. Substituting equation (6) in equation (1a),

$$\frac{Ad(H-h)}{dt} = \frac{awR^2 h}{8\mu L} \quad (7)$$

Substituting for H and h from equations (3) and (4) gives

$$\frac{Ad}{dt} [H_0 \sin \alpha t - h_0 \sin \alpha (t + \epsilon)] = \frac{awR^2 h}{8\mu L}$$

Substituting equation (4) in this expression, and differentiating,

$$A\alpha [H_0 \cos \alpha t - h_0 \cos \alpha (t + \epsilon)] = \frac{awR^2}{8\mu L} \cdot h_0 \sin \alpha (t + \epsilon)$$

$$\text{Inserting the constant } C = \frac{8\mu\alpha AL}{awR^2},$$

$$C [H_0 \cos \alpha t - h_0 (\cos \alpha t \cos \alpha \epsilon - \sin \alpha t \sin \alpha \epsilon)] = h_0 (\sin \alpha t \cos \alpha \epsilon + \cos \alpha t + \sin \alpha \epsilon) \quad (8)$$

When $\cos \alpha t = 0$, $C h_0 \sin \alpha \mathcal{E} = h_0 \cos \alpha \mathcal{E}$ and

$$\tan \alpha \mathcal{E} = \frac{1}{C} \quad (9)$$

When $\cos \alpha t = 1$,

$$C H_0 - C h_0 \cos \alpha \mathcal{E} = h_0 \sin \alpha \mathcal{E}$$

and substitution for C from equation (9) gives

$$\frac{h_0}{H_0} = \cos \alpha \mathcal{E} \quad (10)$$

Equations (9) and (10) determine the constants necessary for computation of the working head at any instant, t . The actual elevation H' in the tide gage well is

$$H' = H - h = H_0' \sin \alpha (t - \theta)$$

Expanding this equation and inserting equations (9) and (10) gives

$$\frac{\frac{H_0'^2}{H_0^2}}{2} = 1 - \frac{h_0^2}{H_0^2} = 1 - \cos^2 \alpha \mathcal{E} = \sin^2 \alpha \mathcal{E}$$

$$\tan \alpha \theta = - \cot \alpha \mathcal{E} \quad (11)$$

Here, $\alpha \mathcal{E} = \tan^{-1} \left(\frac{1}{C} \right)$. The lag in seconds of events on the H' -curve after the corresponding events on the H -curve is the quantity θ .

Both the lag and the reduction in range are seen to be functions of the dimensionless quantity C which may therefore be regarded as the model criterion for the fluctuations in tide gage wells connected to the outside basin by lines which are long in proportion to the maximum velocity head.

Turbulent Flow. - In completely developed turbulent flow in pipes, the head loss is proportional to the square of the velocity. If this quadratic relationship is taken as the definition of "turbulent" flow, then very short pipe lines connecting two reservoirs may be regarded as being in "turbulent flow", even though the flow may actually be laminar, because the working head is utilized almost entirely in creating velocity head which is proportional to V^2 in both turbulent and laminar flow. In other words laminar flow through an orifice or a very short tube should obey the same type of equation (not the same equation) as turbulent flow in a long pipe. The exact length of tube which divides turbulent and laminar flow as defined here cannot be specified exactly but it will depend upon the relationship between the friction head and the velocity head.

The resistance to steady flow through a pipe line connecting two reservoirs is

$$h = K \frac{V^2}{2g} + \frac{fLV^2}{D2g} + \frac{V_e^2}{2g}$$

Here, K represents the entrance, elbow and other losses due to fittings, f is the friction factor corresponding to the velocity V and V_e is the final velocity at exit. The values of K and f depend upon the Reynolds number. The ratio V_e/V equals the coefficient of contraction in the case of an orifice. For a simple pipe connection, $V_e = V$. Inserting the coefficient of contraction and solving for V,

$$V = \sqrt{\frac{2gh}{\frac{\delta}{C_c^2} + K + \frac{fL}{D}}} = \sqrt{\frac{2gh}{M}} \quad (12)$$

Here δ is the velocity head coefficient and C_c is the coefficient of contraction at discharge.

The continuity equation is

$$\frac{Ad(H-h)}{dt} = a \sqrt{\frac{2gh}{M}} \quad (1b)$$

Assuming that h is sinusoidal as before,

$$\frac{Ad}{dt} [H_0 \sin \alpha t - h_0 \sin \alpha(t + \epsilon)] = \pm a \left[\frac{2gh_0}{M} \sin \alpha(t + \epsilon) \right]^{\frac{1}{2}} \quad (13)$$

The solution given by Chapman expresses $[\sin \alpha(t + \epsilon)]^{\frac{1}{2}}$ as a Fourier series which preserves the absolute value of the right hand member.

$$\sin^{\frac{1}{2}} x = a_1 \sin x + a_3 \sin 3x + a_5 \sin 5x + \dots + a_n \sin nx.$$

the values of the constants are:

$$a_1 = 1.113$$

$$a_3 = 1/7 a_1$$

$$a_5 = 5/77 a_1$$

The series becomes

$$[\sin \alpha(t + \epsilon)]^{\frac{1}{2}} = 1.113 [\sin \alpha(t + \epsilon) + 1/7 \sin 3\alpha(t + \epsilon) + 5/77 \sin 5\alpha(t + \epsilon) + \dots]$$

Substituting in equation (13)

$$A \frac{d}{dt} \left[H_0 \sin \alpha t - h_0 \sin \alpha (t + \varepsilon) \right] = 1.113 a \left(\frac{2gh_0}{M} \right)^{\frac{1}{2}} \cdot \left[\sin \alpha (t + \varepsilon) + 1/7 \sin 3 \alpha (t + \varepsilon) + 5/77 \sin 5 \alpha (t + \varepsilon) + \dots \right] \quad (14)$$

Substituting t' for $\alpha (t + \varepsilon)$ and differentiating equation (14),

$$H_0 \cos (t' - \alpha \varepsilon) - h_0 \cos t' = \frac{a}{A} \cdot \frac{1.113}{\alpha} \left(\frac{2gh_0}{M} \right)^{\frac{1}{2}} [\text{series}]$$

$$\text{letting } (2N)^{\frac{1}{2}} = \frac{a}{A} \frac{1.113}{\alpha} \left(\frac{2g}{M} \right)^{\frac{1}{2}}$$

$$H_0 \cos t' \cos \alpha \varepsilon + H \sin t' \alpha \varepsilon = h_0 \cos t' = (2Nh_0)^{\frac{1}{2}} [\text{series}] \quad (15)$$

When $\cos t' = 1$,

$$H_0 \cos \alpha \varepsilon = h_0 \quad (16)$$

and when $\cos t' = 0$,

$$H_0 \sin \alpha \varepsilon = (2Nh_0)^{\frac{1}{2}} \left[1 - 1/7 + 5/77 - + \dots \right] \quad (17)$$

The series converges to unity. Squaring and adding equations (16) and (17)

$$\begin{aligned} H_0^2 \cos^2 \alpha \varepsilon &= h_0^2 + H_0^2 \sin^2 \alpha \varepsilon - 2Nh_0 = 0 \\ H_0^2 - h_0^2 &= 2Nh_0 = 0 \\ h_0 &= (N^2 + H_0^2)^{\frac{1}{2}} - N \end{aligned} \quad (18)$$

Also

$$\cos \alpha \varepsilon = \left(1 + \frac{N^2}{H_0^2} \right)^{\frac{1}{2}} - \frac{N}{H_0} \quad (19)$$

The equations for the range in the tide gage well are:

$$\begin{aligned} H_0'^2 &= H_0^2 - h_0^2 = H_0^2 - \left[(N^2 + H_0^2) - 2N (N^2 + H_0^2)^{\frac{1}{2}} + N^2 \right] \\ &= 2N \left[(N^2 + H_0^2)^{\frac{1}{2}} - N \right] = 2Nh_0 \end{aligned} \quad (20)$$

$$\tan \alpha = - \cot \alpha \varepsilon \quad (21)$$

The dimensionless equations are:

$$\cos \alpha \varepsilon = \left(1 + \frac{N^2}{H_0^2}\right)^{\frac{1}{2}} - \frac{N}{H_0} \quad (22)$$

$$\frac{H_0'}{H_0} = \sqrt{\frac{2N}{H_0} \cos \alpha \varepsilon} \quad (23)$$

$$\tan \alpha \theta = -\cot \alpha \varepsilon \quad (21)$$

The phenomenon is seen to be a function of the dimensionless ratio N/H_0 for the condition assumed.

Acceleration. - In the preceding analysis of the problem, it was assumed that the instantaneous rate of flow in accelerated motion is the same as in steady motion under the same head. This assumption neglects the head necessary to accelerate the flow. The magnitude of the error will now be analyzed for the case of laminar flow.

The equations of motion are,

$$h - h' = \frac{L}{g} \frac{dv}{dt} \quad (24)$$

$$h' = \frac{8\mu LV}{w R^2}$$

$$V = \frac{A}{a} \frac{dH'}{dt} \quad \frac{dV}{dt} = \frac{A}{a} \frac{d^2 H'}{dt^2}$$

$$h - \frac{8 LA}{wa R^2} \frac{dH'}{dt} = \frac{L}{g} \frac{A}{a} \frac{d^2 H'}{dt^2} \quad (24a)$$

From the first approximation,

$$h_0 = H_0 \cos \alpha \varepsilon \quad h = h_0 \sin \alpha (t + \varepsilon)$$

$$H_0' = H_0 \sin \alpha \varepsilon \quad H' = H_0' \sin \alpha (t - \theta)$$

If these relationships are substituted in equation 24a, the equation becomes an inequality since the first approximation was obtained on the assumption that $\frac{dv}{dt} = 0$. Making the substitution

$$\begin{aligned} H_0 \cos \alpha \varepsilon \sin \alpha (t + \varepsilon) - \frac{8\mu LA}{wa R^2} H_0 \sin \alpha \cos \alpha (t - \theta) \\ + \alpha^2 \frac{L}{g} \frac{A}{a} H_0 \sin \alpha \varepsilon \sin \alpha (t - \theta) = 0 \end{aligned} \quad (25)$$

Since $\tan \alpha \theta = -\cot \alpha \varepsilon$, $\alpha \theta + \alpha \varepsilon = \frac{\pi}{2}$ and $\sin \alpha (t + \varepsilon) = \cos \alpha (t - \theta)$.

Substituting this relationship in equation (25) and dividing through by $\cos \alpha (t - \theta)$ gives

$$1 - \frac{8\alpha\mu LA}{waR^2} \tan \alpha \varepsilon + \frac{\alpha^2 LA}{ga} \tan \alpha \varepsilon \tan \alpha (t - \theta) \approx 0$$

The first two members do not vary with time and are equal. The third member then expresses the error resulting from the assumption that the flow at each instant equals the rate of steady flow under the same head. Since $\tan \alpha (t - \theta) \rightarrow \infty$ as $\alpha (t - \theta) \rightarrow \frac{\pi}{2}$, the error is certainly not negligible over some portion of the cycle. Taking the ratio of the acceleration and viscosity terms given by the first approximation,

$$\frac{\frac{\alpha^2 LA}{ga} \tan \alpha \varepsilon \tan \alpha (t - \theta)}{\frac{8\alpha\mu LA}{waR^2} \alpha \varepsilon} = \frac{\alpha R^2}{8\nu} \tan \alpha (t - \theta)$$

An approximate method of determining a limit for α such that the acceleration term will not appreciably affect the results is to hold this ratio below a value n over a certain percentage of the cycle, p . Thus

$$\frac{\alpha R^2}{8\nu} \tan \alpha (t - \theta) < n; \quad \alpha (t - \theta) < p \frac{\pi}{2} \quad (26)$$

For example, to hold the ratio of inertia forces to viscous forces below 0.1 over 75 per cent of the cycle,

$$p \frac{\pi}{2} = 67.5^\circ; \quad \tan p \frac{\pi}{2} = 2.414$$

$$\alpha = \frac{0.1 \times 8 \times \nu}{2.414 R^2}$$

With $\nu = 1.0 \times 10^{-5}$ ft.²/sec. and $R = 0.0112$ ft. (1/8" pipe), $\alpha = 0.033$ radians per second. As these figures for α , R , and ν are approximately those obtained in the experiments reported here, the conditions were such that the inertia forces probably account for some of the discrepancy between the theoretical and experimental results.

The preceding analysis indicates that in long lines the effect of inertia will have a very great effect on the results before the flow becomes turbulent in the usual sense. Referring back to the Reynolds number (Equation 5a)

$$\begin{aligned} R_m &= \frac{2V_m R}{\nu} = \frac{2A}{a} \frac{R}{\nu} \frac{dH}{dt} = \frac{2AR}{a\nu} \alpha H_0 \sin \alpha \varepsilon [\cos \alpha (t - \theta)] v_{\max} \\ &= \frac{2AR\alpha H_0}{a\nu} \sin \alpha \varepsilon \quad t = \theta \end{aligned} \quad (27)$$

The criterion for the maximum value of α is

$$\alpha = \frac{3 n \nu}{R^2 \tan \left(p \frac{\pi}{2} \right)} \quad (26a)$$

If R_m exceeds ≥ 2300 and α from equation (26a) is within reasonable limits for a particular case, then the analysis of the maximum value of α must be modified to fit the case of turbulent flow, starting from equation (24) but using the previous analysis of turbulent flow as the first approximation. On the other hand, if $R_m \leq 2300$ and α given by equation (26a) is far below the value of α for the experimental installation, the connection to the tide gauge well should be modified or its lag and reduction of range should be determined experimentally.

Coefficients. - Practically, the case of turbulent flow is important only in very short lines in which the connection is equivalent to an orifice or short tube. Hence the friction coefficient f in turbulent flow probably enters the equation for M in very few cases. Since the Reynolds number is generally small, the flow through the orifice is laminar and the coefficient C_c is nearly unity. The coefficient K is smaller than in truly turbulent flow but not much is known about the correct value.

Whether a particular problem should be solved by the equations for laminar or turbulent flow depends on the ratio of that portion of the resistance which is proportional to V^2 and that proportional to V . Taking the simple case of laminar flow in a short connection, δ in equation (12) equals 2, $C_c = 1$. The two components of the head h are then

$$2 \frac{V^2}{2g} \text{ and } \frac{8 \mu V}{w R^2}$$

Taking the ratio

$$\frac{8 \mu V}{w R^2} / \frac{2 V^2}{2g} = \frac{16 \nu}{2 V R^2}$$

Inserting the value of V given by the laminar solution

$$V = \frac{A}{a} \frac{dH}{dt} = \frac{\alpha A H_0}{a} \sin \alpha \epsilon \cos \alpha (t - \theta)$$

At $V = V_{\max}$,

$$\frac{16 \nu}{2 V R^2} = \frac{8 a \nu}{\alpha A H_0 R^2 \sin \alpha \epsilon}$$

If this ratio is large as compared with unity, the head necessary to overcome viscous resistance is large as compared with the velocity head and the laminar solution applies. If it turns out to be small as compared with unity, the

turbulent solution probably applies but further analysis of the problem must be made.

Experimental Procedure. - A water level recorder (Steven's Type M) was operated in a gage well placed in the model basin and connected to the water in the basin by pipes and openings of various lengths and diameters. Another identical water level recorder was operated in a second gage well which was completely open at the bottom. The two recorders were synchronized by depressing the pens simultaneously.

Experimental Results. - The problem has been analyzed earlier in this paper on the basis of two types of flow, viz:

Laminar; head loss $\propto v$; long lines

Turbulent; head loss $\propto v^2$; short lines

Under usual conditions the flow probably seldom is turbulent in the strict technical sense but even in laminar flow through a short tube or orifice the effective head is proportional to the square of the velocity and it is in this sense that the term "turbulent" is used here.

On the assumption that the instantaneous rate of flow is the same as during steady flow under the same operating head (inertia neglected) analysis of the problem yields the following results:

(a) Laminar flow.

$$R_m = \frac{2AR \propto H_0}{a \nu} \quad \sin \alpha \epsilon < 2300$$

$$\tan \alpha \epsilon = \frac{1}{C}, \quad C = \frac{8 \mu \propto AL}{a \omega R^2}$$

$$H_0' = H_0 \sin \alpha \epsilon$$

$$\tan \alpha \theta = -\cot \alpha \epsilon$$

$$\theta + \epsilon = \frac{\pi}{2} \text{ or } 90^\circ$$

(b) Turbulent flow.

$$M = \frac{\propto}{C_c^2} + K + \frac{fL}{D}$$

$$N = \left(\frac{1.113 a}{\propto_A} \right)^2 \frac{g}{M}$$

$$\cos \alpha \varepsilon = (1 + \frac{N^2}{H_0^2})^{\frac{1}{2}} = \frac{N}{H_0}$$

$$H_0' = H_0 \sqrt{\frac{2N}{H_0} \cos \alpha \varepsilon}$$

$$\tan \alpha \theta = -\cot \alpha \varepsilon$$

The two types of flow are seen to be functions of the dimensionless quantities C and N/H_0 for laminar and turbulent flow respectively. Figures 1 and 2 are constructed on the basis of these dimensionless variables.

The tide gage measurements were made using straight lengths of standard wrought-iron pipe connected to a 4-inch gage well. The reference measurements were made in a gage well open at the bottom located immediately adjacent to the inlet to the connecting pipe line.

1. Laminar Flow

Tidal Period	Connecting Pipe Diameter		$\frac{A}{a}$	Length (ft.)	Lag (sec)	Measured				Max. Re	C
	Nom.	Actual				$\frac{\alpha \theta}{\pi}$	$\frac{H_0}{ft}$	$\frac{H_0'}{ft.}$	$\frac{H_0'}{H_0}$		
70.8	1/8	.269	223	1	7.2	-.204	.025	.020	0.800	611	.464
	1/8	.269	223	10	15.9	-.450	.025	.004	0.160	178	4.64
	1	1.049	14.76	1	2.1	-.060	.025	.025	1.000	217	.002
138	1/4	.364	123	40	30.9	-.448	.025	.0033	0.132	61	2.87
140.5	1/8	.269	223	0.5	6.5	-.092	.023	.023	1.000	395	.117
	1/8	.269	223	3.0	18.7	-.265	.023	.017	0.740	333	.702
	1/8	.269	223	10.0	29.9	-.425	0.023	.006	0.261	146	2.34
	1/4	.364	123	0.5	1.0	-.015	.027	.027	1.000	306	.035
	1/4	.364	123	3.0	7.2	-.102	.027	.027	1.000	303	.212
	1/4	.364	123	10.0	16.3	-.231	.027	.022	0.815	248	.706

2. Turbulent Flow

Tidal Period	Connecting Hole Diameter		A a	Length (ft.)	Lag (sec)	Measured				N N	N H ₀
	Nom. in.	Actual in.				$-\frac{\alpha \theta}{\pi}$	H ₀ ft.	H ₀ ' ft.	$\frac{H_0'}{H_0}$		
70.8	1/8	.267	227	0	2.3	-.065	.025	.025	1.000	.0816	3.27
	1	1.004	16.08	0	0.9	-.025	.026	.026	1.000	16.3	627
138.0	1/16	.0625	4140	0	33.8	-.490	.0242	.0035	.146	.00093	.0384
140.5	1/8	.267	227	0	6.5	-.092	.023	.023	1.000	.322	14.0
	1/4	.350	132	0	1.0	-.014	.027	.027	1.000	.948	35.1

Experimental Accuracy

The elevations on the tide graphs can be measured to + 0.001 foot. The time intervals can be read to at least + 2 seconds and perhaps even more accurately. The maximum range of H₀ was about 0.025 feet and H₀' went as low as 0.003 so that the error in elevation measurements was roughly + 4% and + 35% with the smaller error occurring with the larger ranges. As the lag increased, the percentage error in the measured values decreased but was always greater than + 5%. The remarks on accuracy are made as an explanation of the scattering of the experimental points, shown on Figures 1 and 2.

Sample Calculation

(a) A tide gage is to be located in a 4-inch well and connected to the main tidal basin by 40 feet of $\frac{1}{4}$ inch standard wrought iron pipe.

$$H_0 = 0.025 \text{ ft.}$$

$$\frac{A}{a} = 123$$

$$L = 40 \text{ ft.}$$

$$\mu = 2.35 \times 10^{-5} \text{ lb. sec.}$$

$$R = 0.0152 \text{ ft.}$$

$$T = 138.0 \text{ sec.}$$

$$w = 62.4 \text{ lbs./cu. ft.}$$

$$\alpha = 0.0447 \text{ rad./sec.}$$

$$C = \frac{8\mu\alpha LA}{wR^2a} = \frac{8 \times 2.35 \times .0447 \times 10^{-5} \times 40 \times 123}{62.4 \times (.0152)^2} = 2.87$$

$$-\tan \alpha \theta = C; \quad \alpha \theta = -70^\circ - 40^\circ$$

$$\theta = -70.7 \times \frac{140.5}{360} = 27.5 \text{ sec (measured 31 + 2 sec.)}$$

$$H_0' = H_0 \cos \alpha \theta = 0.025 \times 0.331 = 0.008 \text{ ft.}$$

$$\text{Measured } H_0' = 0.003 \pm 0.001 \text{ ft.}$$

(b) Same conditions as in (a) except that connecting line is made up of 10 ft. of 1/2 inch wrought-iron pipe.

$$H_0 = 0.023 \text{ ft.} \quad \mu = 2.3 \times 10^{-5} \text{ lb. sec./ft. sq.}$$

$$L = 10 \text{ ft.} \quad T = 140.5 \text{ sec.}$$

$$R = 0.0111 \text{ ft.} \quad \frac{A}{a} = 223$$

$$w = 62.4 \text{ lbs./cu. ft.} \quad \alpha = \frac{2\pi}{140.5} = 0.0447 \text{ radians/sec.}$$

$$C = \frac{8\mu\alpha AL}{waR^2} = \frac{8 \times 2.35 \times 10^{-5} \times 0.0447 \times 223 \times 10}{62.4 \times (0.0111)^2} = 2.34$$

$$\alpha\theta = -67^\circ - 40'$$

$$\theta = 67.7 \times \frac{140.5}{360} = 26.4 \text{ sec. (measured } 30 + 2 \text{ sec.)}$$

$$H_0' = 0.023 \times 0.380 = 0.0088 \text{ ft.}$$

$$\text{Measured } H_0' = 0.006 \pm 0.001 \text{ ft.}$$

A tide gage is to be placed in a gage well 4 inches in diameter. The connection is a 1/8" cylindrical hole drilled through the tube wall.

$$(a) \quad H_0 = 0.025 \quad A/a = 223$$

$$L = 0 \text{ (approx.)} \quad \alpha = \frac{2\pi}{70.8} = 0.0888 \text{ rad./sec.}$$

$$a = 0.057 \text{ sq. in.}$$

$$T = 70.8 \text{ sec.}$$

$$M = 1.2$$

$$N = \frac{1}{2} \left(\frac{a}{A} \frac{1.113}{\alpha} \right)^2 \frac{2g}{M} = \frac{1}{2} \left(\frac{1.113}{223 \times 0.0888} \right)^2 \frac{64.4}{1.2} = 0.088$$

$$h_0 = 0.089 - 0.088 = 0.001 \text{ ft.}$$

The measured value of h_0 was of the order of 0.001 ft., the precision of the gage being ± 0.001 ft. and no reduction in range being observable.

$$(b) \quad H_0 = 0.024 \text{ ft.} \quad \alpha = 0.0447 \text{ sec.}$$

$$L = 0 \quad C_0 = 1$$

$$\frac{A}{a} = 4150 \quad f = 0$$

$$T = 140.5 \text{ sec.} \quad K = 0.2$$

$$M = 1.2$$

$$N = \frac{1}{2} \left(\frac{1.113}{4150 \times 0.0447} \right)^2 \quad \frac{64.4}{1.2} = 0.001$$

$$h_0 = 0.023 \text{ ft.}$$

$$H_0' = \sqrt{2Nh_0} = 0.0068 \text{ ft. (measured} = 0.003 \pm .001 \text{ ft)}$$

$$\cos \alpha \epsilon = \frac{h_0}{H_0} = \frac{0.023}{0.024} = 0.957 ; \alpha \epsilon = 16^\circ - 50'$$

$$\tan \alpha \theta = - \cot \alpha \epsilon$$

$$\alpha \theta = - 73^\circ = 10' \text{ or } -1.28 \text{ radians}$$

$$\theta = - \frac{1.28}{2\pi} \cdot 140.5 = 29 \text{ seconds}$$

$$\text{Measured } \theta = 34 \pm \text{ sec.}$$

Referring back to the same calculation on page 34, (Laminar flow, Example (a))

$$\frac{8a \nu}{\alpha A H_0 R^2 \sin \alpha \epsilon} = \frac{8 \times 1.05 \times 10^{-5}}{0.0447 \times 123 \times .0125 \times (0.0152)^2 \times .331} = 16$$

The viscous head is large as compared with the velocity head.

Conclusions

On the basis of the comparisons made it is believed that the theoretical curves may be relied upon more than experimental results because the error in the experimental results is relatively great. This statement is made on the assumption that the various criteria developed in the theory are satisfied.

* * * * *

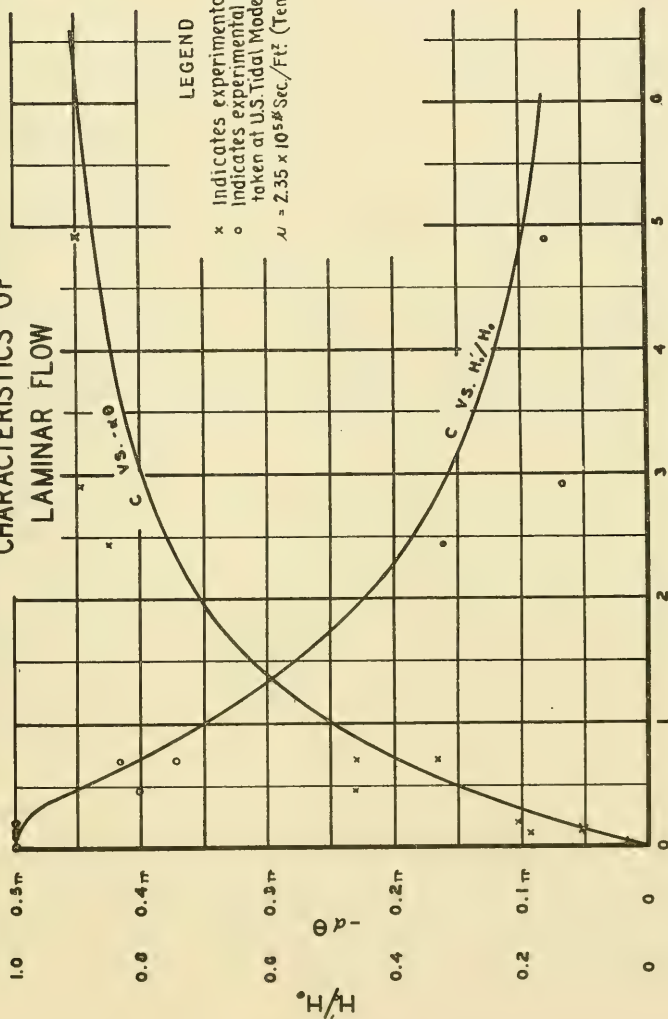
APPENDIX I

SYMBOLS

- H = elevation of primary tide above mean level at time t
- H_0 = half the amplitude of the primary tide curve
- H' = elevation of tide in gage well at any time t
- H_0' = half the amplitude of tide curve in well
- h = effective head at any time t
- h_0 = half the amplitude of effective head
- t = time
- $t' = \alpha (t + \epsilon)$
- ϵ = lead of effective head in seconds
- θ = lag of tide curve in gage well in seconds
- T = tidal period
- n = ratio of inertia to viscous head
- p = percentage of cycle during which ratio of inertia to viscous heads is less than n
- $\alpha = 2\pi / T$
- a = area of connecting line
- A = area of tide-gage well
- R = radius of connecting pipe
- D = diameter of connecting line
- L = length of connecting line
- C = characteristic of gage well and connecting line for laminar flow
- N = characteristic of gage well and connecting line for "turbulent" flow.
- C_0 = coefficient of contraction at discharge from connecting line
- k = coefficient of loss at bends, contractions, inlet, etc.
- f = friction factor for connecting line
- M = overall resistance coefficient for turbulent flow. (Eq. 12)

V = velocity in connecting line
 V_e = average mean velocity at discharge from connecting line
 δ = velocity head coefficient
 μ = absolute viscosity
 ν = kinematic viscosity
 w = weight of fluid per cubic foot
 R_m = Reynold's number for flow in line at maximum velocity
 R_e = Reynolds number at any velocity V

LAG IN TIDE GAGE WELLS

CHARACTERISTICS OF
LAMINAR FLOW

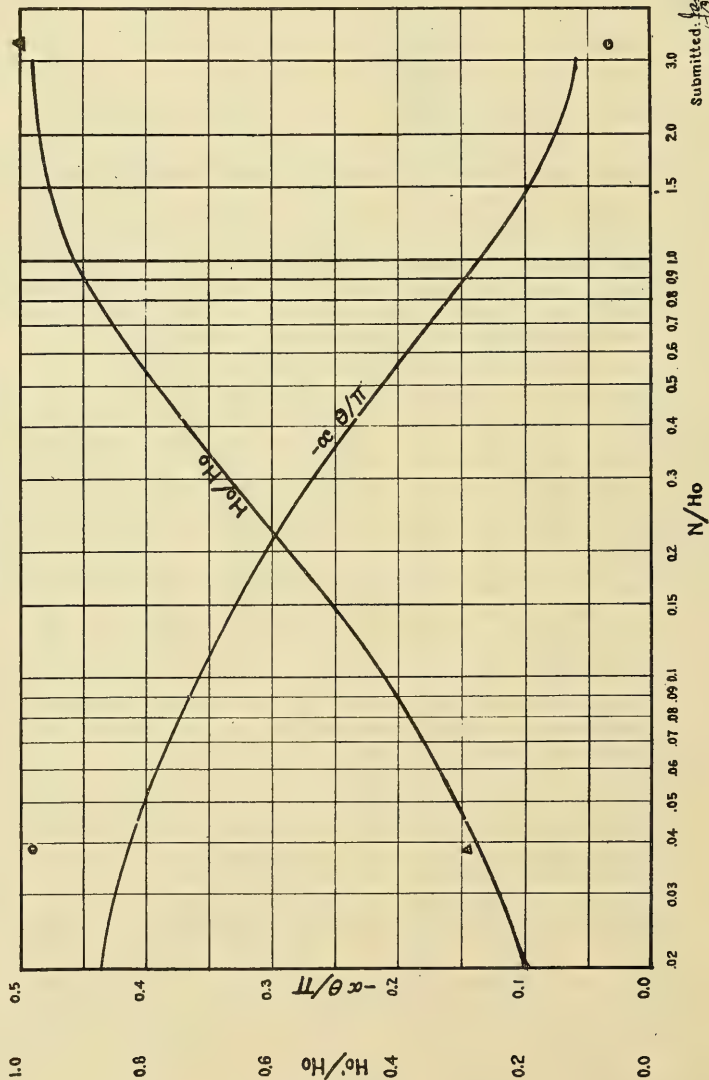
$$C = \frac{A \delta B \lambda L}{\alpha \omega R^2}$$

Submitted: *Blanchard & Bin*
 Approved: *Manuel D. Brien*
 Drawn by: GCP
 5-2-36

LAG IN TIDE GAGE WELLS

CHARACTERISTICS OF

TURBULENT FLOW



Submitted: *James D. Mahan*
 Approved: *James D. Mahan*
 Consulting Engineer.
 Drawn By J.D.M. - 6-11-36.

BEACH EROSION STUDIES

The principal types of beach erosion reports of studies at specific localities are the following:

- a. Cooperative studies (authorization by the Chief of Engineers in accordance with Section 2, River and Harbor Act approved on 3 July 1930).
- b. Preliminary examinations and surveys (Congressional authorization by reference to locality by name).
- c. Reports on shore line changes which may result from improvements of the entrances at the mouths of rivers and inlets (Section 5, Public Law No. 409, 74th Congress).
- d. Reports on shore protection of Federal property (authorization by the Chief of Engineers).

Of these types of studies, cooperative beach erosion studies are the type most frequently made when a community desires investigation of its particular problem. As these studies have, consequently greater general interest, information concerning studies of specific localities contained in these quarterly bulletins will be confined to cooperative studies. Information about other types of studies can be obtained upon inquiry to this office.

Cooperative studies of beach erosion are studies made by the Corps of Engineers in cooperation with appropriate agencies of the various States by authority of Section 2, of the River and Harbor Act approved 3 July 1930. By executive ruling the cost of these studies is divided equally between the United States and the cooperating agency. Information concerning the initiation of a cooperative study may be obtained from any District Engineers. After a report on a cooperative study has been transmitted to Congress, a summary thereof is included in the next issue of this bulletin. A list of completed cooperative studies and of those now in progress follows.

SUMMARY OF REPORT TRANSMITTED TO CONGRESS

STATE OF OHIO - LAKE COUNTY

The area studied is located in Lake County on the south shore of Lake Erie from 20 to 30 miles east of Cleveland, Ohio. It lies between the mouths of the Chagrin and Grand Rivers, a distance of about 10 miles. Fairport Harbor, which has been improved by the United States for navigation, is located at the mouth of the Grand River, the east limit of the study area.

Lake County had a population of about 50,000 in 1940. The principal center of population is the city of Painesville, which had a population of about 12,000. The property along the shore line of the study area has been developed mainly for private residential and recreational purposes. The principal summer colony is at Mentor Headlands. The population of the area as a whole is not increased greatly by summer visitors. Inland areas are devoted mainly to agricultural uses. The shore is publicly owned at the Painesville Water Works and at Mentor Township Park. The latter is used for recreational purposes. The State of Ohio intends to acquire lakefront and upland property between the Painesville Water Works and Fairport Harbor for public beach and park development. There are wide beaches on existing and proposed public ownership frontage.

The shore area immediately east of the mouth of Chagrin River is subject to pollution by sewage discharged untreated into that stream. Plans are being made for proper treatment of this sewage. No apparent hazards from sewage contamination exist in the eastern portion of the study area.

The shore line of the study area consists principally of eroding bluffs of clay, silt and sand fronted by narrow beaches of sand and gravel. Analysis of samples of bluff material indicated that in general approximately 27 per cent of the material is suitable for beach building. Rapid erosion of the bluffs makes available a considerable volume of beach material. A wide beach has formed naturally on both sides of Chagrin River. West of Mentor and Fairport Harbors wide beaches have formed by accretion caused by the harbor structures.

Miscellaneous groins, walls, and bulkheads have been constructed in an attempt to prevent erosion of the shore. Shore groins have generally caused minor accretion on their west sides and have reduced recession of the bluffs to some extent. The pronounced accretion west of the harbor structures and the accretion west of short groins indicate a marked eastward predominance of littoral drift. The purpose of the study was to determine the best methods of preventing further erosion and of restoring and creating new beaches, with emphasis on determining effective and economical methods of shore protection and on the possibilities of development and improvement of publicly owned park and beach areas.

The district engineer considered the desires of the cooperating agency, studied the sources and movement of beach material, the changes in the shore line and offshore bottom, the effects of winds, waves, ice, and storms, the effects of existing structures, and developed four general plans for protecting and improving the shores of the study areas. He concluded that the most economical and practical general plan of protection consists in grading and draining of the bluffs, revetment of the toe of the slope, and maintenance of relatively narrow beaches by means of short groins. He concluded further that the beaches at the proposed State Park west of Fairport Harbor and other publicly owned shore frontage require no improvement at this time. He recommended that owners of private property adopt one of the four proposed plans of improvement best suited to the desired utilization of their shore front property. Since no improvements or protection of publicly owned property in the area were considered necessary at the time, he recommended no

Federal participation in the cost of any proposed improvements. The division engineer concurred in the conclusions and recommendations of the district engineer.

The Board carefully considered the reports of the reporting officers. It concurred generally in their views and recommendations, subject to the following comments.

The Beach Erosion Board concluded that protection and improvement of the shore by means of long groins would involve too long a period of time before adequate filling of the groin system by natural processes would be effected. The plan could be supplemented by provision of artificial fill, but no need exists for beaches of the width that would be provided in this manner and such a supplemented plan probably lacks economic justification. Grading of the bluffs and armoring of the toe by a rock sea wall or cellular steel pile bulkhead would provide positive protection for the bluff, but would inhibit the use of the beach for recreational purposes. The Board concurred in the opinion of the reporting officers that an economical and practicable general plan of protection comprises grading and draining of the bluff, revetting the toe of the slope and maintaining a narrow beach by means of relatively short groins. This plan provides protection for the bluff as well as an improved beach for recreational purposes. Filling of the groin system depends on supply of material from erosion of the bluffs within the study area and also those along the shore west of Chagrin River. If the entire shore within the area is protected under this plan, filling will be dependent on the supply of material from west of the area and will therefore be slow. More rapid filling could be effected if bluffs within the area where protection is not essential are permitted to erode and supply material. The Board considered the possibility that this plan may be beyond the means of some private owners of shore frontage. Where this condition exists and no structures are located so close to the top of the bluff as to necessitate positive protection against recession of the bluff, a less costly modification of this plan may be used. This modification consists of short groins a minimum of about 100 feet in length from the toe of the bluff on a spacing of about 100 feet. Under this modified plan the slope armoring would be omitted so the erosion of the bluff would supply material to fill the groin system. The groin would operate to retard erosion of the beach and the beach might be expected to build up to protect the toe of the bluff. In order to retain the beach at the toe of the bluff, the inner ends of the groins should be above the height reached by the highest waves. It appears that a height of 10 feet above low water datum would meet this requirement. The top of the groin should parallel the natural slope of the bottom and adequate provision should be made against flanking of the groins. The Board called attention to the desirability of coordinated action by owners to protect a stretch of frontage under this plan. Although an individual owner of a short frontage could protect his property by a groin near his east property line, the protection would not be permanent. Erosion would eventually flank his property and necessitate further protective measures.

The Beach Erosion Board recommended that private owners adopt one of the four plans of protection and improvement proposed by the district engineer or that described in the previous paragraph, selecting that most

suitable to the desired use of their shore frontage. As existing Federal law includes no policy for Federal assistance in the cost of protecting privately owned shores, and as no protection or improvement of publicly owned property is considered necessary under present conditions, no Federal participation in the cost of any of the proposed work was recommended by the Beach Erosion Board at this time.

* * * * *

COMPLETED COOPERATIVE BEACH EROSION STUDIES

<u>Location</u>	<u>Completed</u>	<u>Published in</u> <u>House Doc. Congress</u>	
MAINE			
Old Orchard Beach	20 Sep 35		
NEW HAMPSHIRE			
Hampton Beach	15 Jul 32		
MASSACHUSETTS			
South Shore of Cape Cod (Pt. Gammon to Chatham)	26 Aug 41		
Salisbury Beach	26 Aug 41		
Winthrop Beach	12 Sep 47	764	80
Lynn-Nahant Beach	20 Jan 50		
Revere Beach	12 Jan 50		
Nantasket Beach	12 Jan 50		
Quincy Shore	2 May 50		
RHODE ISLAND			
South Shore (Towns of Narragansett, South Kingstown, Charlestown & Westerly)	4 Dec 48	490	81
CONNECTICUT			
Compo Beach, Westport	18 Apr 35	239	74
Hawk's Nest Beach, Old Lyme	21 Jun 39		
Ash Creek to Saugatuck River	29 Apr 49	454	81
Hammonasset River to East River	29 Apr 49		
NEW YORK			
Jacob Riis Park, Long Island	16 Dec 35	397	74
Orchard Beach, Pelham Bay, Bronx	30 Aug 37	450	75
Niagara County	27 Jun 42	271	78
South Shore of Long Island	6 Aug 46		

NEW JERSEY

Manasquan Inlet & Adjacent			
Beaches	15 May 36	71	75
Atlantic City	11 Jul 49	538	81

VIRGINIA

Willoughby Spit, Norfolk	20 Nov 37	482	75
Colonial Beach, Potomac River	24 Jan 49	333	81

NORTH CAROLINA

Fort Fisher	10 Nov 31	204	72
Wrightsville Beach	2 Jan 34	218	73
Kitty Hawk, Nags Head & Oregon Inlet	1 Mar 35	155	74
State of North Carolina	22 May 47	763	80

SOUTH CAROLINA

Folly Beach	31 Jan 35	156	74
-------------	-----------	-----	----

GEORGIA

St. Simon Island	18 Mar 40	820	76
------------------	-----------	-----	----

FLORIDA

Blind Pass (Boca Ciega)	1 Feb 37	187	75
Miami Beach	1 Feb 37	169	75
Hollywood Beach	28 Apr 37	253	75
Daytona Beach	15 Mar 38	571	75
Bakers Haulover Inlet	21 May 45	527	79
Anna Maria & Longboat Keys	12 Feb 47	760	80
Jupiter Island	13 Feb 47	765	80
Palm Beach (1)	13 Feb 47	772	80

MISSISSIPPI

Hancock County	3 Apr 42		
Harrison County - Initial	15 Mar 44		
Harrison County - Supplement	16 Feb 48	682	80

LOUISIANA

Grand Isle	28 Jul 36	92	75
------------	-----------	----	----

TEXAS

Galveston	10 May 34	400	73
Galveston Bay, Harris County	31 Jul 34	74	74

- (1) A cooperative study of experimental steel pile groins was also made, under which methods of improvement were recommended in an interim report dated 19 Sep 1940. Final report on experimental groins was published in 1948 as Technical Memo. No. 10 of the Beach Erosion Board.

CALIFORNIA

Santa Barbara - Initial	15 Jan 38	552	75
Supplement	18 Feb 42		
Final	22 May 47	761	80
Ballona Creek & San Gabriel River (Partial)	11 May 38		
Orange County	10 Jan 40	637	76
Long Beach	3 Apr 42	636	77
Mission Beach	4 Nov 42		
Coronado Beach	4 Apr 41		

PENNSYLVANIA

Presque Isle Peninsula, Erie (Interim)	3 Apr 42		
(Final)	23 Aug 49		

OHIO

Erie County - Vicinity of Huron	26 Aug 41	220	79
Lake Erie Shore - Michigan Line to Marblehead	30 Oct 44	177	79
Cities of Cleveland & Lakewood	22 Mar 48	502	81
Lake County	22 Nov 49		

ILLINOIS

State of Illinois	8 Jun 50		
-------------------	----------	--	--

WISCONSIN

Milwaukee County	21 May 45	526	79
------------------	-----------	-----	----

PUERTO RICO

Punta Las Marias, San Juan	5 Aug 47	769	80
----------------------------	----------	-----	----

COOPERATIVE BEACH EROSION STUDIES IN PROGRESS

NEW HAMPSHIRE

HAMPTON BEACH. Cooperative Agency; New Hampshire Shore and Beach Preservation and Development Commission.

Problem: To determine the best method of preventing further erosion and of stabilizing and restoring the beaches, also to determine the extent of Federal aid in any proposed plans of protection and improvement.

CONNECTICUT

STATE OF CONNECTICUT. Cooperating Agency: State of Connecticut
(Acting through the Flood Control and Water Policy Commission).

Problem: To determine the most suitable methods of stabilizing and improving the shore line. Sections of the coast will be studied in order of priority as requested by the cooperating agency until the entire coast is included.

NEW YORK

JONES BEACH. Cooperating Agency: Long Island State Parks Commission

Problem: To determine behavior of the shore during a 12-months cycle, including study of littoral drift, wave refraction and movement of artificial sand supply between Fire Island and Jones Inlets.

NEW JERSEY

OCEAN CITY. Cooperating Agency: City of Ocean City.

Problem: To determine the causes of erosion or accretion and the effect of previously constructed groins and structures, and to recommend remedial measures to prevent further erosion and to restore the beaches.

VIRGINIA

VIRGINIA BEACH. Cooperating Agency: Town of Virginia Beach.

Problem: To determine the methods for the improvement and protection of the beach and existing concrete sea wall.

SOUTH CAROLINA

STATE OF SOUTH CAROLINA. Cooperating Agency: State Highway Department.

Problem: To determine the best method of preventing erosion, stabilizing and improving the beaches.

FLORIDA

PINELLAS COUNTY. Cooperating Agency: Board of County Commissioners.

Problem: To determine the best methods of preventing further recession of the gulf shore line, stabilizing the gulf shores of certain passes, and widening certain beaches within the study area.

LOUISIANA

LAKE PONTCHARTRAIN. Cooperating Agency: Board of Levee Commissioners, Orleans Levee District.

Problem: To determine the best method of effecting necessary repairs to the existing sea wall and the desirability of building an artificial beach to provide protection to the wall and also to provide additional recreational beach area.

TEXAS

GALVESTON COUNTY. Cooperating Agency: County Commissioners Court of Galveston County.

Problem: To determine the best method of providing a permanent beach and the necessity for further protection or extending the sea wall within the area bounded by the Galveston South Jetty and Eight Mile Road.

To determine the most practicable and economical method of preventing or retarding bank recession on the shore of Galveston Bay between April Fool Point and Kemah.

CALIFORNIA

STATE OF CALIFORNIA. Cooperating Agency: Division of Beaches and Parks, State of California.

Problem: To conduct a study of the problems of beach erosion and shore protection along the entire coast of California. The initial studies are to be made in the Ventura-Port Huneme area and the Santa Monica area.

WISCONSIN

RACINE COUNTY. Cooperating Agency: Racine County.

Problem: To prevent erosion by waves and currents, and to determine the most suitable methods for protection, restoration and development of beaches.

KENOSHA. Cooperating Agency: City of Kenosha.

Problem: To determine the best method of shore protection and beach erosion control.

OHIO

STATE OF OHIO. Cooperating Agency: State of Ohio (Acting through the Superintendent of Public Works).

Problem: To determine the best method of preventing further erosion of and stabilizing existing beaches, of restoring and creating new beaches, and appropriate locations for the development of recreational facilities by the State along the Lake Erie shore line.

TERRITORY OF HAWAII

WAIKIKI BEACH. Cooperating Agency: Board of Harbor Commissioners, Territory of Hawaii.

WAIMEA, HANAPEFE, KAUAI.

Problem: To determine the most suitable method of preventing erosion, and of increasing the usable recreational beach area, and to determine the extent of Federal aid in effecting the desired improvement.

BEACH EROSION LITERATURE

There are listed below some recent acquisitions to the Board's library which are considered to be of general interest. Copies of these publications can be obtained on 30-day loan by interested official agencies.

"Measurement of Ocean Waves," R. G. Folsome, Trans. American Geophysical Union, Vol. 30, No. 5, October 1949

Laboratory and field investigations demonstrate that theoretical pressure ratios for pressure wave recorders are correct to within ten to 20 per cent. The theoretically corrected measured wave heights in shallow water are low. The University of California Mark III and Mark V wave recorders are described and typical records from the latter are reproduced. The limitations of the available information on spar buoy damping disc systems for deep water wave measurements are summarized.

"Hydrodynamic Forces on a Rough Wall," Hans Albert Einstein, Review of Modern Physics, Vol. 21, No. 3, July 1949.

The dynamic forces which a turbulent flow exerts on the individual protrusions of a rough wall have been measured. It was found that even in the case of an extremely high relative roughness, the drag force on the protrusions may be determined from the logarithmic friction laws. A very significant lift force which tends to pull the the protrusions away from the wall and into the flow was measured. This lift force was divided into two parts: A constant average value and a random fluctuation superimposed over the average. A careful statistical analysis showed that the frequency of different pressures or lift forces at the wall follows very accurately the normal error law. This fact seems to indicate that in the description of turbulence near a rough wall, the pressures must be regarded as the primary influence, not the velocities.

"Scale Effects in Hydraulic Models Involving Wave Motion," J. W. Johnson, Trans., American Geophysical Union, Vol. 30, No. 4, August 1949.

A summary of the various instances where wave theories have been verified experimentally is presented briefly. A more detailed discussion is given for those problems where models built to several scales have permitted an evaluation of the effect of scale in the application of data from hydraulic models.

"An Analysis of Data From Wave Recorders on the Pacific Coast of the United States," R. L. Wiegsl, Trans. American Geophysical Union, Vol. 30, No. 5, October 1949.

The wave heights and period from wave recorders installed off Point Sur, California, and Hecta Head, Oregon, have been compared for the period from April 1947 to June 1948. These data, together with some obtained at Point Arguello, California, showed that the ratio of the

maximum wave recorded each day to the average of the highest ten per cent for that day was 1.46 and the ratio of the average of the highest ten per cent to the average of the highest one-third of the waves each day was 1.29 for any place along this section of the Pacific Coast. It was also found that the average period of the swells (that is, excluding the local storm "chop") was 12 seconds.

"A Theory of the Hydraulic Injector," Duilio Citrini (Saggio Di Teoria Dell'Iniettore Idraulico), Memorie e Studi dell'Istituto di Idraulica del Politecnico di Milano, No. 72, 1948

The analysis of the motion in the mixing-room of a hydraulic injector (jet pump) is developed, for incompressible fluids, without using the hypotheses, usually but incorrectly admitted, of constant pressure head and negligible losses of head. The increase of the pressure head along the device is evaluated. For the losses of head an approximate analytical expression is given.

The efficiencies of the device for different situations are examined and examples of practical calculation are given.

The theory is developed for the injector with cylindrical mixing-room, but successively the case of a convergent mixing-room is too examined, according to a type proposed by Rateau; the comparison shows that the cylindrical type furnishes in general a higher efficiency.

"Critical Comparison of Measurements of the Vertical Distribution of Wind On the Sea," Hans-Ulrich Roll, (Vergleichende Betrachtung und Kritik von Windprofilmessungen auf See) Annales of Meteorology, March-April 1949.

A comparison of the measurements of the vertical distribution of wind in the lowest atmospheric layer above the sea-surface leads to the following results: The profiles measured by Montgomery, Shoulejkin, and Roll agree satisfactorily, disturbances of the field of wind speed having been avoided as far as possible. The measurements of Wust, Bruch and of the Altair voyage, however, show departures in the sense of greater roughness parameters above the level of 50 - 100 cm. By a discussion of the possible source of error and by comparative measurements it was made probable that these differences are not due to fundamental causes but to inaccuracies in the technique of measuring. In particular it appeared that the measurements of wind on the bow of an anchored ship may yield results that are 50 - 100 cm/s too high.

"The Application of Conformal Transformations of Ocean Wave Refraction Problems," New York University, Leon S. Pocinki, April 1950.

If the bottom contours are considered straight and parallel, and if the usual assumptions are made, a simple differential equation for the orthogonals can be derived from Snell's Law. Solutions to the equation are easily obtained. Conformal transformations may be applied to change the contours in the complex Z plane into several different configurations in a complex W plane. In each case an orthogonal in the Z plane maps into a new orthogonal in the W plane and thus a solution to the problem of wave refraction over the contour configuration in the W plane is obtained. An expression for K_3 is then derived from the orthogonal pattern. Cases treated include a circular island, a reef and a bay with parabolic contours, and twin islands.

